NON DARCY FLOW THROUGH POROUS MEDIA

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By

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To

MY ELDER BROTHER

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CERTIFICATE

Certified that this work, 'Non-Darcy Flow Through Porous Media' has been carried out by Shri H.S. Niranjan, under my supervision and that this work has not been submitted elsewhere for a degree.

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CONTENTS

NOTATIONS

A = cross sectional area of inlet tube

A_x = X-area of conical permeameter at distance x from the base

a, b = coefficient of Forchheimer's equation.

B = constant for conical permeameter

C = constant

C_0 = 1/S, S = Coefficient of micro roughness

C_1, C_2 = constant

C_3 = variable depending on size, shape, porosity

D = dia. of cylindrical permeameter (for nonlinear flow experiment)

D_1 = dia. of pore channel

d = nominal size of gravel

ds = equivalent spherical dia. given by $\frac{R_H \, 6}{e}$

e = void ratio

f = constant

F = friction factor given by $\frac{g \, i \, ds \, n^3}{v^2(1-n)G(\text{wall})(0.1067)}$

F_1 = '' '' '' $\frac{d \, i}{2 \rho v^2}$

F_2 = '' '' '' $\frac{2 \, g \, ds}{L_1 \, v}$

F_3 = Friction factor given by $\frac{1}{Re} + 1.0$

F_4 = " " " $\frac{1}{Re} + 0.55 \sqrt{C_3}$

g = acceleration due to gravity

G(wall)= wall effect

h_f = head loss

h = head difference between inlet and exist at any instant

H_1, H_2 = heads of water at inlet of the conical permeameter for time interval t_1 and t_2.

i, I = hydraulic gradient

i_x = hydraulic gradient in a element at distance x from base.

K = coefficient of permeability

K_s = specific permeability

L = height of conical permeameter

L_1 = length of pore channel

m = constant

n = percentage porosity

p = hydraustatic pressure

q = discharge through conical permeameter

R = hydrautic mean radius given by $\frac{nd}{6\ (1-n)^3}$

R'	=	constant depending on shape and surface roughness of gravel.
Re	=	Reynold's number given by $\frac{v\,ds\,\rho\,(0.6)}{\mu\,(1-n)}$
R_H	=	hydraulic mean radius given by $\frac{e}{\text{surface area / volume}}$
r_*	=	hydraulic mean radius
r_1, r_2	=	bottom and top radii of conical permeameter
s	=	taper of the sides of conical permeameter
u, v	=	velocity in x and y direction
V_v	=	seepage velocity
V, v	=	discharge velocity
μ	=	dynamic viscosity of fluid
ν	=	kinematic viscosity of fluid
ρ	=	mass density of fluid
γ	=	weight density of fluid
α_1, β_1	=	constant.

ABSTRACT

For flow through soils, Darcy's law has been generally found to be valid. However, it is so only when viscous forces predominate over inertial forces. In case of flow through coarser material like sand and gravel the inertial forces and viscous forces are of the same order of magnitude and deviations from the Darcy's law are observed. In such cases the equation suggested by Forchheimeter ($i = av + b\ v^2$) or Missbach ($i = Cv^m$) are used for relating the hydraulic gradient with velocity. In both the equations, the coefficients (a, b) and (C, m) depends on the physical characteristics of the porous media and the flow conditions.

A review of the literature shows that considerable theoretical and experimental work has been done on non-linear flow problems. But no efforts seems to have been made to relate the coefficients a and b of the Forchheimer equation, specially the coefficient 'b' with the physical characteristics of the media. Therefore, the present study is mainly intended to relate the coefficient 'b' with size, porosity, shape and roughness of the gravel. Since past available data are insufficient for this study therefore, experiments were carried out on the gravel in the range of sizes from 0.318 to 4.62 cm and glass balls from 1.3 cm to 2.5 cm.

Results shows that 'b' varies inversly with size and porosity of gravel. Two relationships have been established between the coefficient 'b' and the nominal size and another between coefficient 'b' and size, porosity, surface roughness and shape of the gravel. A new non-dimensional parameter has been developed and its variation with porosity has been presented in the graph. Also, the variation between Reynolds number and friction factor has been shown in another graph.

A conical permeameter has been developed so that it generate a range of hydraulic gradients, sands and gravels can be tested in this such that an average value of K can be obtained for the range of gradient existing. Analytical expressions are obtained for head loss and discharge velocity through the conical permeameter. Typical results for K of fine sand obtained from the conical and cylindrical permeameter are found to compare well.

CHAPTER 1

INTRODUCTION

Problems concerning flow of water through soils have become one of the most interesting and important field of study among the Civil Engineering subjects. This is because of new achievements in the fields of Soil Mechanics and Hydraulics Engineering. Whenever any huge structures like dams and other hydraulic structures are constructed, one meets with problems like uplift pressure, seepage, stability of slopes and many problems related to the flow of water in soils. In ground water engineering one comes across flow of water in to wells, trenches, etc. Hence much attention is being paid towards such problem.

In the past as early as 1856 Henry Darcy first studied the problem of water flow in compacted sands experimentally and put forward the linear relation ship between head loss and discharge velocity, which is popularly known as Darcy's law. Later on other investigators such as Dupuit, Povlovsky etc. applied the Darcy's law in various seepage problems.

However it has been recognised that there are two principal types of flow in soils.

1. Laminar flow: In which fluid particles travels along definite paths which never cross each other. For this flow Darcy law is valid.

2. Turbulent Flow: In which paths are irregular, twisting, crossing and recrossing each other at random.

For the most cases of flow through soils of practical interest, the flow velocities and particle size are small enough for laminar conditions to exist and Darcy's law hold good. To discuss the fundamental approach to discribe flow through porous media, it is necessary to mention the approach suggested by Philip (1957) and developed by Watson (1963). In this, use is made of Navier - Stokes' equations of motion which are derived from classical hydrodynamical principles. For newtonian fluids, steady motion, incompressible flow and neglecting body forces, it can be shown (Kay - 1957) that for a two dimensional medium.

$$\nu \nabla^2 u - \frac{1}{\rho} \frac{\partial p}{\partial x} = u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} \qquad (1.1)$$

$$\nu \nabla^2 v - \frac{1}{\rho} \frac{\partial p}{\partial x} = u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y} \qquad (1.2)$$

where u, v = velocity in x,y directions

ρ = mass density

p = hydrostatic pressure which would occur with equilibrium under gravity at the point considered.

ν = Kinamatic viscosity

In these equations the first term represents the viscous forces and the right hand side the inertial terms.

It should be emphasized, however, that Navier - Stoke's equation using mean flow values are restricted in use of laminar flow condition and on the basis of classical hydrodynamics, it follows that Darcy's law is at least only a good approximation at low Reynoulds number since the inertial terms are always present.

As the flow velocities, particle size and Reynold's number increase and the laminar conditions do not exist, Darcy's law becomes increasingly in-accurate and another flow law is required. The reason for the non-linearty is the increasing significance of inertia forces as recognised by Tek (1957), Green and Duwez (1951), Hubbert (1956), Watson (1963). Tek and O'Neill (1965) have suggested three general regimes of flow through porous media.

1. Darcy flow regime: In this viscous forces predominate, inertial forces are insignificant and Darcy's law is closely followd.

2. Laminar non Darcy flow regime: At higher Reynold numbers where inertial forces become significant as well as viscous forces the flow may still be laminar, but non-linear and Darcy's law is no longer obeyed.

3. Turbulent flow regime: At very high Reynold's number, inertial forces become large as compared to viscous forces, turbulence sets in and is initially localized in large voids, fractures, cavities etc.

The range of validity of Darcy's law has been a matter of discussion. Watson (1963) fixes this limit between 0.2 to 5.0 while Scheidigger (1960) fixes as 0.1 to 75.0. This large variation in upper limit is because of the factors like, size, shape, roughness and porosity of particle are not considered.

As obvious from category-2-laminar non Darcy regime, the hydraulic gradient (i) is no longer directly proportional to the flow velocity (v). Therefore some relationship between (i) and (v), other than Darcy's law, is necessary for such flow conditions. Forehheimer (1901) and Missbach (1937) have suggested a polynomial and exponental forms of relationships between (i) and (v). The relationships are:

$$i = av + bv^2 \qquad \text{(Forcheimer)} \qquad (1.3)$$

$$i = c\,v^m \qquad \text{(Missbach)} \qquad (1.4)$$

where i is hydraulic gradient,

v is macrovalocity,

a and b are constants which depend on size, shape, surface roughness of the gravel pack and the flow conditions.

c and m are also constants depending on flow conditions. By analogy with pipe flow, it could be expected the exponent 'm' will vary from 1.0 for laminar flow to 2.0 for fully turbulent flow.

Nonlinear laminar flow and turbulent flow through porous media is encountered in several field situation, e.g.

a - flow through rock fill dams

b - flow through filters

c - flow is areas adjacent to a well

d - flow in rock fissures.

A review of literature (Chapter -2), shows that considerable work has been done on nonlinear flow problems. The experimental work has been mainly carried out to determine the coefficients a, b, and c, m of Forchheimer's Missbach equations and no attempt has been made to relate these coefficient specially a, b with the physical characteristics of the porous media. Therefore in this work, Chapter-3 is mainly intended to study the coefficients a and b of Forchheimer's equation with a view to relate them to the size, porosity, roughness of the media.

In chapter 4 an attempt has been made to develop a permeameter to generate a range of hydraulic gradients and flow velocities. The properties determined from such a permeameter could be expected to give average values of K for the range of gradients or velocities generated. Analytical expressions useful for this study are given there in.

Concluding remarks are given in chapter 5 and are followed by a set of references.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL:

A brief review of literature on nonlinear flow through porous media is presented here. The entire work can be divided into two categories. Thus,

(1) Theoretical Work:

(a) Study of the upper limit of Darcy's law, and determining the laws governing high velocity flow through porous media.

(b) Application of the above theory to the various field situations and relating the flow characteristics of the porous media to the coefficients of the governing laws (Head loss equations).

(2) Experimental Work:

Determination of the values of the coefficients of the head loss equations, study of onset of the turbulence, determination of the various limits of flow regimes, study of effect of convergence of stream lines and wall effect, study of the relation between friction factor and Reynolds number, study of seepage problems by electrical analogue.

2.2 THEORETICAL WORK:

(a) Hubbert (1940) reports that Darcy's experiments were carried out on a well compacted filter medium consisting mainly of sand but with proportion of large particles. Most of the tests were confined to linear flow but he recognised that there was an upper limit to the validity of this law. He defined the limit in terms of velocity approximately 10 to 11 cms/sec. for this material. He has not mentioned the size and energy of compaction given for his material.

Taylor (1948) calculated that for a unit hydraulic gradient the maximum diameter of the uniform sand in which the laminar flow will occure is 0.5 mm.

The accepted method of expressing the limitation of Darcy's law is to determine the Reynold's number at the point where departure from the linear relationship is first noted. Watson (1963) has carried out a computer solution of the Navier Stroke equations for two dimensional flow to determine the values of Reynolds number at which the permeability becomes noticeably affected by inertial terms. The results shows a rapid increase in the inertial effect between Reynolds number of 0.2 to 5.0. Scheidegger (1960) and Todd (1959) quote the range as from 0.1 to 75.0

and 1.0 to 10 respectively.

Thus the large variation in the Reynolds number reflects the defficiency of the usual form of parameter in accounting the variables involved in flow, such as size, shape roughness and porosity of the particles. Because of this defficiency it was felt necessary to do the experimental work in the context of the present study.

The generally accepted form of head loss equation in non Darcy regime, and which has been suggested by Forehheimer (1901) is as follows:-

$$i = av + bv^2 \qquad (2.1)$$

In which a and b are constants depending on the physical characteristics of the media and flow conditions such as Reynolds number. This equation has been experimentally investigated by Volker (1968), Ahmed and Sundada (1969), O'Neill, Parkin, (1970), Todd and Tyagi (1970), Ranganadha Rao and Suresh (1970). By deduction from Navier - Stoke's equation Ahmed and Sundada (1969), showed that the governing head loss equation is of the form suggested by Forchheimer.

Another form of equation suggested by Missbach (1937) is:

$$i = cv^m \qquad (2.2)$$

In which c is constant depending on media and flow condition and m is an exponent which is 1.0 for laminar flow and 2.0 for complete turbulent. Above equation has been supported by Anandakrishnan and Varadarajalu (1963), Parkin, Trollope, Lowson (1966).

Wilkins (1955) using wide range of aggregate sizes, derived the equation of the form:

$$V_v = C_1 \mu^{\alpha_1} r_*^{\beta_1} i^{n_1} \qquad (2.3)$$

where V_v = seepage velocity

n_1 = $(1/m)$

r_* = hydraulic mean radius

μ = dynamic viscosity of fluid

C_1, α_1, β_1, are constants.

The value of C_1 varied from 32.9 for crushed stone to 46.5 for marbles in inch units.

It may be noted that Irmay (1958) has remarked that although for any type of flow, the nonlinear acceleration terms in the Navier - Strokes equation may be neglected at small velocities but it is only for straight pipes that these terms are identically zero at higher velocities. For flow through porous media their presence in the equation will have the effect of causing the resistance - velocity

relation to be nonlinear.

2.2(b) Ahmed and Sundada (1969) derived the equation from the Navier - Stroke's equation of the form

$$i = \frac{\mu}{\rho g K} v + \frac{1}{g\sqrt{CK}} v^2 \qquad (2.4)$$

on comparing this equation with Forchheimer's eqn. (2.1), it has been found that

$$a = \frac{\mu}{\rho g K} \quad \text{and} \quad b = \frac{1}{g\sqrt{CK}}, \quad \text{where}$$

C = constant

g = acceleration due to gravity

K = coefficient of permeability

μ = dynamic viscosity

ρ = mass density of water.

Izbash and Lelleeve (1971) derived the formula of the form

$$i = \frac{\beta \mu}{R^2 g} v + \frac{1}{C_0^2 R} v^2 \qquad (2.5)$$

on comparing with Forchheimer's equation (2.1) gives

$$a = \frac{\beta \mu}{R^2 g} \quad \text{and} \quad b = \frac{1}{C_0^2 R}$$

where $C_o = 1/S$, S = coefficient of microroughness.

R = hydraulic mean radius

β = coefficient depending on cross sectional shape

μ = dynamic viscosity

Izbash and Khaldre (1959) studied the problem of turbulent seepage through fill material. Author has presented the results in form of formulae, tables and graphs for the seepage through partially or fully submerged river closures. These closure were made by simply dumping the gravels or cribs and which were triangular in cross-section and having large voids.

Mc Corquodale and Ng (1969) applied the finite-element method to solution of non Darcy's flow with a free surface problems.

O'Kat (1969), Volker (1969) also applied the finite element method to study nonlinear flow through rock fill dam with sheet piles.

Madhav and Subramanya (1971) studied the nonlinear flow problems arising from confined and unconfined flow into wells and trenches.

Subramanya, Valsangkar and Rao (1971) have studied the unconfined flow over an impervious boundary and flow towards a fully penetrating well by using Forchheimer's equation.

2.3 EXPERIMENTAL WORK:

Many investigators have carried out the experimental work and came out with interesting and useful results which are presented here in brief.

Todd (1959) found in his experimental investigation that in case of flow through porous media, a particular type of relation was exhibited between Reynolds number and friction factor. He used the formula $F_1 = \frac{d\ i}{2\ \rho\ v^2}$ to calculate friction factor in which d = size of gravel, ρ = mass density of water and v is the velocity of water.

Parkin (1963) made extensive studies on a model dam in 4' wide flume and found the method of gradually varied flow is satisfactory to determine the water surface profile.

Anandakrishnan and Varadajlu (1963), Dudgeon (1964) obtained the values of c and m of Missbach equation and the values of a and b Forchheimer's equation for different grades of sand and different sizes of gravels respectively.

Dudgeon (1966) obtained the coefficients C and m of Missbach equation. On the basis of his results he developed an expression $F_2 i = F_2 \frac{v^2}{2gds}$, in which F_2 is friction factor given by $\frac{2g\ ds}{L_1\ v}$, ds is the equivalent

spherical dia., v is velocity, g is acceleration due to to gravity and L_1 is the length of channel.

Dudgeon (1967) determined the wall effect in the permeameter. He used Missbach equation for his investigations. It was found the exponent m was 1.81 for (wall) outer zone and 1.82 for inner zone; the velocity ratio between wall zone and inner zone was 1.06 which shows that permeability in the wall zone is higher than in the inner zone, the ratio of coefficient C between inner zone and wall zone was approximately 4.0.

Robincurtis (1967) has studied the flow through rock fill banks and analysed the stability of slope against seepage.

Wright (1969) carried out experiments to detect and measure turbulence and to observe the effect of convergence of stream lines on Reynolds number vs friction factor graph. It was found that flow could be classified in four regimes.

1. A laminar regime in which velocity remains unchanged at any moment and Darcy's law is valid.
2. A steady inertial regime in which velocity remains unchanged but Darcy's law is not valid.
3. A turbulent transition regime in which velocity fluctuates at any point with increasing but regularly and head loss approximately depends on square of velocity.

4. A fully turbulent regime in which all parts of flow are turbulent, velocity fluctuates randomly about a mean. The head loss now depends on square of velocity.

Volker (1969) obtained the values of a,b and c, m of Forchheimer's and Missbach equations respectively by lab studies of model of gravel banks in an open flume.

Ahmed and Sundada (1969) showed on the basis of the results of 18 tests that friction factor (F) and Reynolds number (Re) bear a relationship

$$F_3 = \frac{1}{Re} + 1.0 \qquad (2.6)$$

He used Forchheimer's equation for the investigation.

Rubic (1969) made extensive study on nonlinear flow problems and presented an electrical analogue model for the solution of such problems.

Ranganadha Rao and Suresh (1970) have investigated the values of coefficient a and b for Forchheimer's equation for different sizes of gravels. It was concluded the relationship given by

$$F = \frac{1}{Re} + 1.0 \qquad (2.7)$$

Proves only the validity of Forchheimer's equation but can not prove the correctness of the values a and b calculated from hydraulic measurements.

Ward (1970) carried out experiments and obtained a relation between friction factor (F) and Reynolds number (Re) which follows:

$$F_4 = \frac{1}{Re} + 0.55 \sqrt{C_3} \qquad (2.8)$$

which is similar to **eqn. (2.6) obtained by** Ahmed & Sundada previsouly, except for $0.55\sqrt{C_3}$ in place of 1.0 where C_3 is a variable depending on size, shape, surface roughness, porosity.

Todd and Tyagi (1970) obtained values of a and b of Forchheimers equation and based on the data a graph between friction factor and Reynolds number was presented which **Cont**-radicts the findings of Ahmed and Sundada (1969), Ward (1970), Ranganadha Rao and Suresh(1970).

Izbash and Leleeve (1971) has defined two limits of Reynolds number Re^{I}_{lim} First boundary Reynolds number, above which results obtained by assuming pure laminar flow conditions leads to an error of 15 per cent.

Re^{I}_{lim} ~~second boundary Reynolds number, above which results obtained by~~ using square law leads to ~~error less than 15 per cent~~.

Re^{II}_{lim} second boundary Reynolds number, above which results obtained by using square law leads to error less than 15 per cent.

With these definitions new laws for heads loss have been defined

$$I^{I}_{lim} = 216 \frac{\mu^2 \beta}{g d^3} Re^{I}_{lim} \left(\frac{1-n}{n}\right)^3 \qquad (2.9)$$

$$I^{II}_{lim} = 216 \frac{\mu^2}{C_o d^3} (Re^{II}_{lim})^2 \left(\frac{1-n}{n}\right)^3 \qquad (2.10)$$

where I = hydraulic gradient

μ = dynamic viscosity

g = acceleration due to gravity

n = porosity

d = size of gravel

C_o = 1/s, where s is coefficient of microroughness

β = coefficient depends on cross-sectional shape.

Subramanya and Rao (1971) obtained relationship, based on experimental work between Forchheimer's coefficients a and b and the specific per-meability (ks) which are -

$$a\ Ks = 10^{-5} \qquad (2.11)$$

$$b\ Ks = 1.42 \times 10^{-4} \qquad (2.12)$$

The specific permeability (ks) was obtained by

$$ks = (k/\gamma) \tag{2.13}$$

where γ is ~~density~~ viscosity of water and

k is the permeability.

The above two relations are valid in the range of Reynolds number from 10 to 200 which is one of the limitations to use the relations. Since size shape, surface roughness and porosity, which are important factors in the flow of water through porous media are not included in the relationship discussed, it was felt necessary to study these parameters and to get a relationship between them.

CHAPTER 3

STUDY OF NONLINEAR FLOW PARAMETERS

3.1 GENERAL

It is clear from the literature review in chapter 2, that even-though some studies have been made on the non Darcy flow through soil, no effort seems to have been devoted ~~to~~ in relating the coefficient a and b, especially the later, of the Forchheimer's equation, with the physical characteristics of the porous media . The available data is insufficient in detail to establish a relation between the parameters of interest and hence, this study.

3.2 EXPERIMENTAL SET-UP:

A detailed diagram of the experimental setup is shown in Fig. 3.1. Continuous supply of water is provided by the overhead tank which maintains a constant head, the permeameter is 15 cm in dia. and 37.5 cm long with a number of pressure tappings on it. These tappings are located in two rows 90° apart along the length of the permeameter and are staggered. This arrangement of pressure tappings gives a continuous measurement of the hydraulic gradient along the length of the permeameter. Any local variations or inadmissible values are easily located. A perforated plate is placed at the bottom of the permeameter to support the material. A valve

is provided at the junction of the permeameter and the delivery pipe to regulate the flow. The delivery pipe, which has a slit in the base rests in a measuring tank of size 90 cm x 75 cm. This measuring tank is fitted with a 90° v-notch to measure the discharge. A drain adjacent to the measuring tank collects the water which in turn flows to a sump and a centrifugal pump lifts the water to the overhead tank to complete the cycle. The measuring tank is provided with a device to read the water level in it. Pressure difference between any two tappings is read on a mercury manometer. A wooden mesh is provided near the delivery pipe in the measuring tank to dissipate all the energy of the discharged water.

Material for investigation has been selected from ~~the~~ gravel and glass balls. The sizes of gravel are 0.318, 0.636, 1.115, 1.75, 2.38, 3.33 and 4.62 cms. The sizes of glass balls are 1.3, 1.7, 2.0 and 2.5 cms. Material used is uniform in size.

It was decided to test each size of the material at three different porosities, very dense, very loose and a porosity in between the two extremes.

3.3 PACKING OF THE MATERIAL:

Small sizes of the material were packed by a hammer (5 kg.) to get the required porosity. The bigger sizes ($\geq$ 1.5 cm)

were packed by hand to get the required porosity. While packing the material, following precautions were taken.

(1) Material was packed uniformly

(2) Top of the packing was made level and average depth was measured to get the height of packing.

(3) If hammer has been used to compact the material than care was taken to see that the material was not crushed by the hammer blows.

(4) If material has been packed loosely by hand than proper interlocking of individual gravel was assured.

(5) All attempts have been made to get three distinct porosities.

3.4 PREPARATION OF THE SETUP:

The permeameter, filled with the material to be tested was fixed in position. Both top and bottom joints were made leak proof. Mercury levels in the manometer were checked initially and scales adjusted to read similar readings in both the limbs. All the pressure tappings were closed except the one which has been connected to the manometer. The by pass valve of the manometer was opened. The initial reading of water level in the measuring tank

was noted. The pump was started and the water was allowed to flow through the permeameter. In the beginning it was noticed that air bubbles were coming out from the tube. As the bubbling stopped the other tube was connected to the required pressure tapping . All the connections were checked and assured to be leakproof.

3.5 METHOD OF OBSERVATIONS:

Observation were taken for decreasing and increasing flow rates. For any flow rate 9 manometer readings were taken for 9 different combinations of the pressure tappings. The water level in the measuring tank was noted. Thus 9 pressure tapping readings and one water level reading completes one set of observations for one flow rate. With the help of regulating valve flow was slightly reduced such that water level in the measuring tank was reduced by approximately 2 cms. Then another set of observation was recorded. Thus 5 sets were taken for decreasing flow rates and 4 sets for increasing flow rates. At the end, the temperature of the water was recorded. Each test was repeated till reproducible results were obtained.

During observations the following precautions were taken:

1. If mercury level in the manometer was found to be oscillating, the average value was recorded.

2. By-pass valve of the manometer was operated very carefully. It is necessary to do so, specially in the case of low pressure differences.

3. All efforts were made to dissipate all the energy of water in the measuring tank. This helps in reading accurately the water level.

4. Bottom supporting mesh of the permeameter was replaced by a coarser mesh before testing larger sizes of the gravel.

5. Only silt free clear water was allowed to flow through the permeameter.

3.6 RESULTS AND DISCUSSION:

From the observations taken, the discharge in cc/sec., velocity (v) in cm/sec. and the average gradient (i) were obtained. Then the values of i/v, Reynold's number (Re) and friction factor (F) were also calculated.

Coefficient a and b were obtained by a graphical method and were verified by a statistical fit.

Graphical Method:-

Points were plotted on a graph with i/v and v taken on vertical and horizontal axes respectively. These points

were found to follow a straight line. The intercept of this straight line on the vertical axes and its slope (multiplied by scale ratio) give the coefficients a and b respectively.

Statistical Fit:-

A computer programme was prepared to fit a straight line to the observed data using the method of least squares, and the two parameters a and b obtained.

Experimental values of 'a' and 'b' as obtained are shown in Table (3.1).

The variation of i/v and v is shown in Fig. 3.4 through 3.10 for the different sizes of gravel and Fig. 3.11 for all sizes of glass balls. These plots confirm the validity of the Forchheimer's equation for the ranges of hydraulic gradients and velocities used. The flow can be classified as 'Steady Inertial' as per Wright (1968). The linear fit shown is extended to meet the Y-axis but it is believed that the actual curve would be different from this extended line due to the existence of laminar flow regime where Darcy's law is valid.

The coefficient 'a' obtained by extending the fit straight line fit would be slightly smaller than the actual value. It is obvious that the values of 'a' for each size, increases with decreasing porosity 'n' (or void ratio-e).

This is a consistant result as 'a' is equivalent to the inverse of the coefficient of permeability and the latter should decrease with decrease in 'n' or 'e'. Jagadeesha Rao (1971) attempted to corretate 'a' with the media characteristics and gave the following:

$$'a'\ K_s = 10^{-5} \qquad (3,1)$$

where K_s is the specific permeability.

The nonlinear parameter 'b' increases with decreasing values of porosity for all sizes of gravel and glass balls. For the smallest size of gravel used (size 0.318 cm), the porosity could not be varied much and hence the slope of $1/v$ vs v curves varied by small amount only. For sizes which could be tested over larger ranges of porosity the slope and hence the coefficient 'b' varied over a greater range.

By analogy with pipe flow, the inertial loss can be written as

$$h_f = \frac{f\ L_1\ v^2}{2g\ D_1} \quad \text{or} \quad \frac{h_f}{L_1} = \frac{f\ v^2}{2g\ D_1} \qquad (3.2)$$

where h_f is head loss, f is a constant depending on Reynolds number, L_1 and D_1 are the length and dia of flow and v is the velocity. Comparing this with second

term of the Forchheimer's equation ($i = av + bv^2$), one can write

$$b = \frac{\text{constant}}{D_1} \qquad (3.3)$$

where D_1 is the size of capillary of the porous media. Since this D_1 decreases with decreasing porosity and particle size, one could anticipate to get higher values of 'b'. The same feature is observed in all the cases.

Fig. 3.11 depicts the results of all sizes of glass balls tested. With the glass balls it was not possible to get more than two porosities for each size. However, the trends are very similar to the observed with gravel.

The dotted lines show the Reynolds number contours on some of the curves drawn for the sizes 0.318, 1.75 and 4.62 cm of the gravel. These contours were drawn by first writing the corresponding values of the Reynolds number on each point then interpolating. It has been found that the contours are nearly vertical in the case of 0.318 cm size and inclined towards right side with decreasing slope for increasing discharge and the size of the gravel.

The influence of the pore channel size which itself depends on particle size, shape and porosity of the medium, onthe nonlinear parameter is studied further. If the variations in 'b' are related to the particle size, the

observation made earlier that 'b' decreases with an increasing particle size and porosity, gets confirmed.

The available data (Coeff. b) from Dudgeon (1964), Ahmed and Sundada (1969), Ranganadha Rao and Suresh (1970) and the data of this work have been plotted with the size of the gravel on a log-log plot (Fig. 3.12). It has been found that all points lie closely along a straight line. The equation of the straight line was found to be

$$\text{Log}_{10}^{d} = 0.712 \log_{10}\left(\frac{1}{b}\right) - 1.237 \qquad (3.4)$$

equation (3.4) is further simplified to

$$b = \frac{.0185}{d^{1.41}} \qquad (3.5)$$

The equation 3.4 or 3.5 does not include the parameters other than size. Hence this equation gives only the approximate variation of coefficient 'b' with the size of the gravel. However, Fig. 3.13 shows the same variation on natural scale.

Fig. 3.14 shows the variation of 'b' with porosity for different sizes of gravel and glass balls. The effect of increasing b-value with decreasing porosity is obvious. An important aspect i.e. the shape and the roughness of the pore channel can be seen from this figure. In case of glass balls the channels are bound by smooth spherical

surfaces while in case of gravel the particles are angular and their surface is rough. Between the two factors shape and surface roughness, the later is supposed to have a dominating influence. The 'b' values are smaller for glass balls as their surfaces are relatively smooth.

Coefficient 'b' has been related to the parameters like - equivalent spherical dia 'ds', porosity 'n', surface roughness and shape of gravel and glass balls. An emperical formula relating the above parameters and the coefficient 'b' was found to be

$$b = \frac{1}{(ds\ n)^{R'}} \tag{3.6}$$

where R' is a parameter for the steady inertial flows. Since it was not possible to measure surface roughness and shape factor separately their effects have been included in one parameter R'. The experimental values of R' for angular and highly rough surfaces like gravel is 1.0 and for spherical and smooth surfaces like glass balls is 1.25. Therefore for intermediate rough surfaces and shapes like river gravels, the parameter R' may be interpolated. A similar emperical formula has been suggested by Aravin (1965), in which power of (ds n) is 2.0 for turbulent flows, irrespective of shape and roughness of the particles.

A new non-dimensional parameter has been obtained by multiplying the coefficient 'b' by the acceleration due to gravity (g) and the equivalent spherical dia (ds). This new parameter has been studied in relation with the other non-dimensional parameter, i.e. porosity 'n'. Fig. 3.15 shows the variation of b.g.ds with'n which also clearly dipicts the influence of surface roughness and shape of gravel on value of 'b'. Hence this graph can be used to predict 'b' for the known values of porosity and the equivalent spherical dia (ds).

Fi . 3.16 illustrates the F vs Re variation. In this graph the points are found to be spread in a small range. This spread is due to the large variation in the porosities in which the each material has been tested graph shows two curves i.e. one for gravel and another for glass balls. However, the trend of the graph is similar to the graph obtained by the other investigator. Curve is nonlinear for Re greater than 8.0.

TABLE 3.1

EXPERIMENTAL VALUES OF 'a' and 'b'

Type of Material	Nominal size (cm)	percentage porosity	'a' Sec/cm.	'b' Sec^2/cm^2
Gravel	0.318	42.0	0.288	0.093
		41.6	0.300	0.101
		41.2	0.300	0.103
		40.2	0.310	0.107
''	0.638	43.5	0.06	0.042
		40.8	0.08	0.048
		36.0	0.10	0.067
''	1.115	43.0	0.016	0.026
		38.0	0.076	0.041
		34.0	0.160	0.054
''	1.750	46.5	0.01	0.0102
		41.5	0.02	0.0105
		36.0	0.035	0.0173
''	2.38	44.7	0.005	0.004
		40.5	0.005	0.00824
		35.5	0.007	0.0148
''	3.330	50.0	0.008	0.0021
		46.6	0.01	0.0029
		43.0	0.055	0.0040

Contd. next page

Contd. Table 3.1

Type of Material	Nominal size (cm)	percentage Porosity	'a' Sec/cm	'b' Sec^2/cm^2
Gravel	4.62	50.0	0.002	0.0010
		46.5	0.004	0.0019
		41.6	0.028	0.00372
Glass Balls	1.30	35.5	0.0324	0.0081
''	1.70	36.7	0.0133	0.0066
		36.1	0.0293	0.00677
''	2.00	38.3	0.0192	0.00448
		36.7	0.0278	0.00525
''	2.50	39.2	0.0140	0.00275
		38.1	0.0209	0.00347

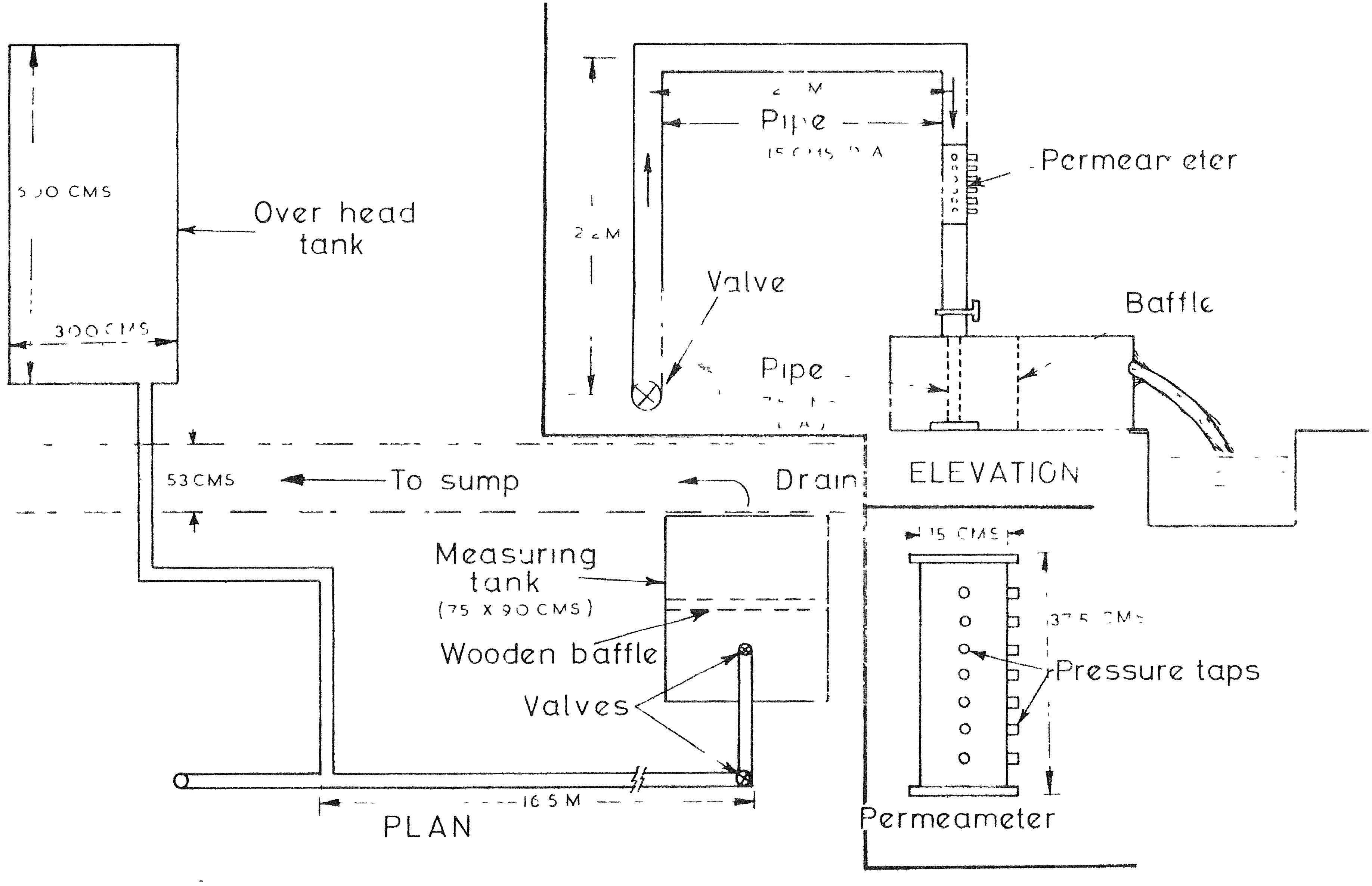
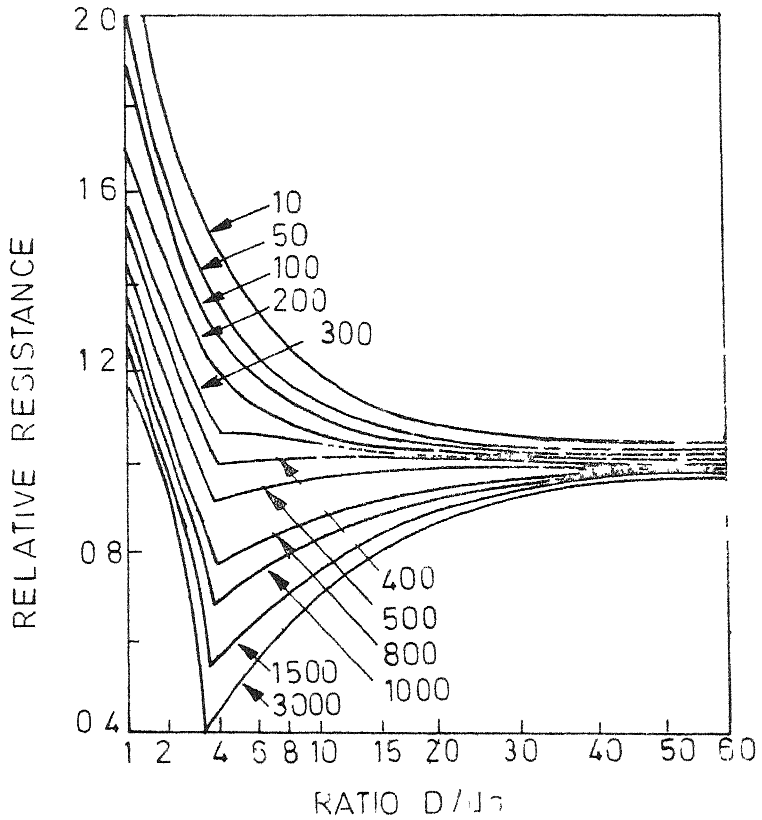


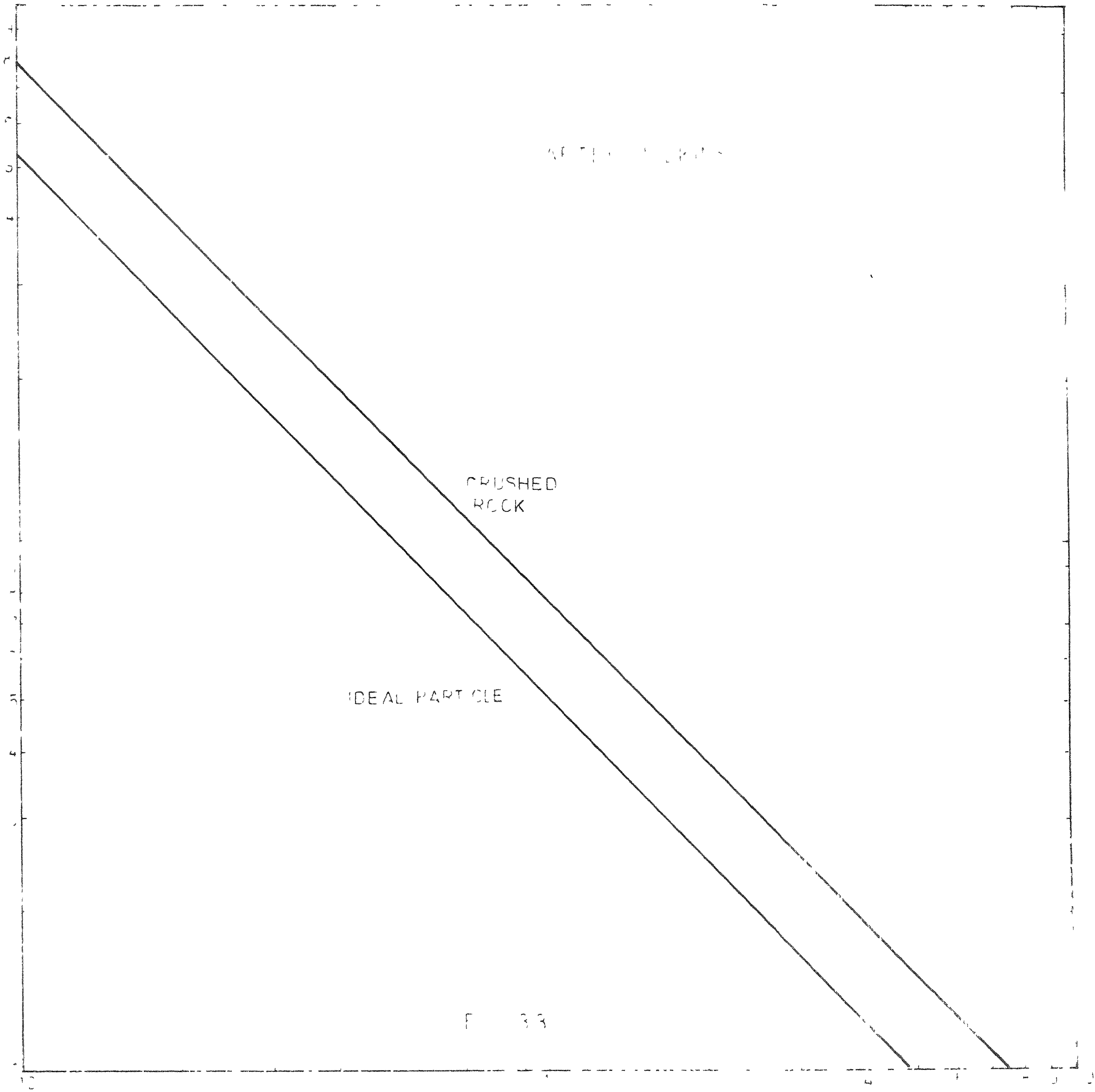
FIG -SCHEMATIC DIAGRAM OF THE EXPERIMENTAL SET-UP

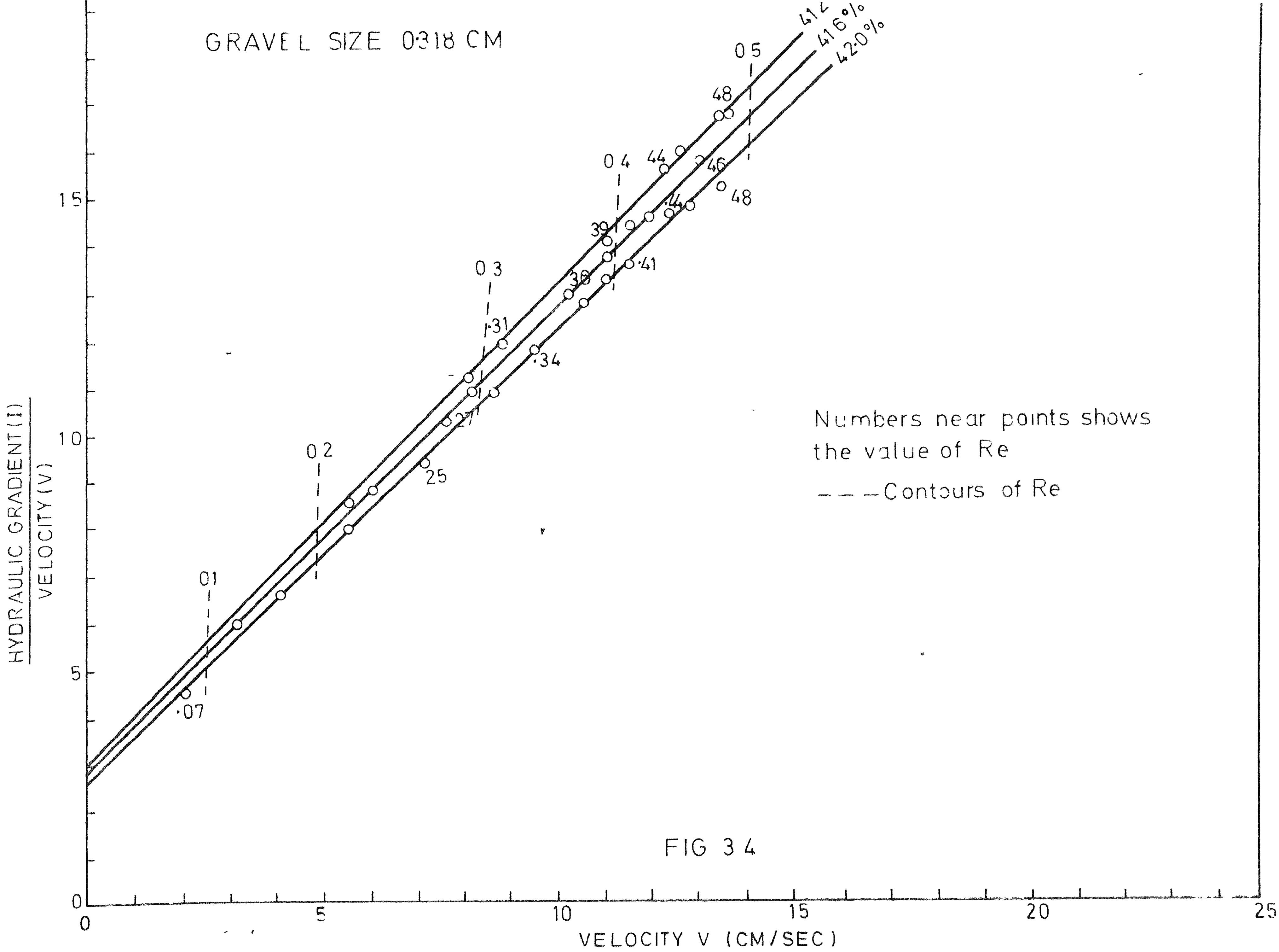


CURVE OF WALL EFFECT (Spherical materi)

REF ROSE AND RIZK, PROCE OF I M E 1949

FIG 3·2


CRUSHED
ROCK
IDEAL PARTICLE



GRAVEL SIZE 0·318 CM
41·2 %
41·6 %
42·0 %
Numbers near points shows the value of Re
– – –Contours of Re
HYDRAULIC GRADIENT (I) / VELOCITY (V)
VELOCITY V (CM/SEC)
0·1
0·2
0·3
0·4
0·5
·07
25
·27
·31
·34
36
39
·41
44
·44
46
48
48
0
5
10
15
20
25

FIG 3 4

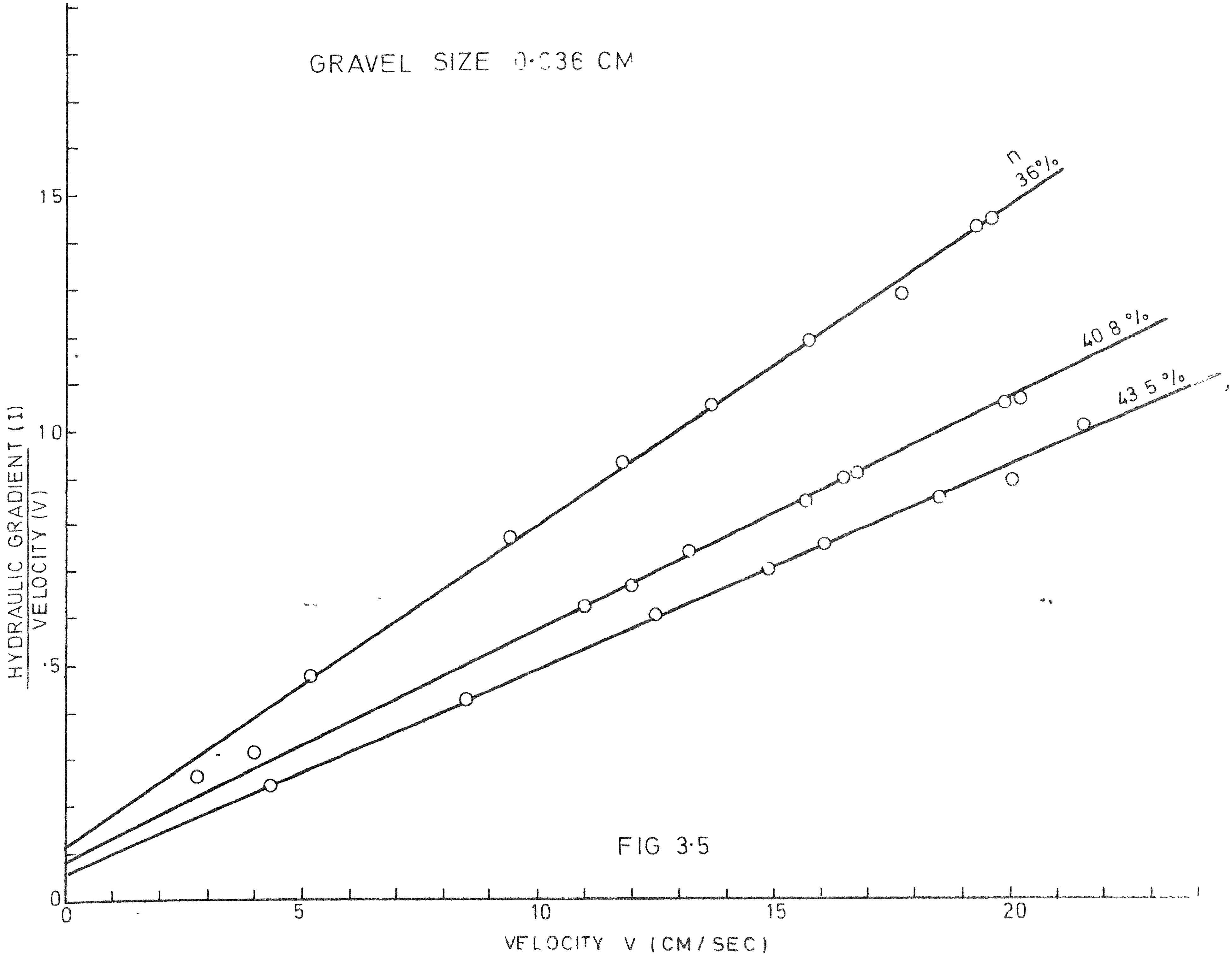
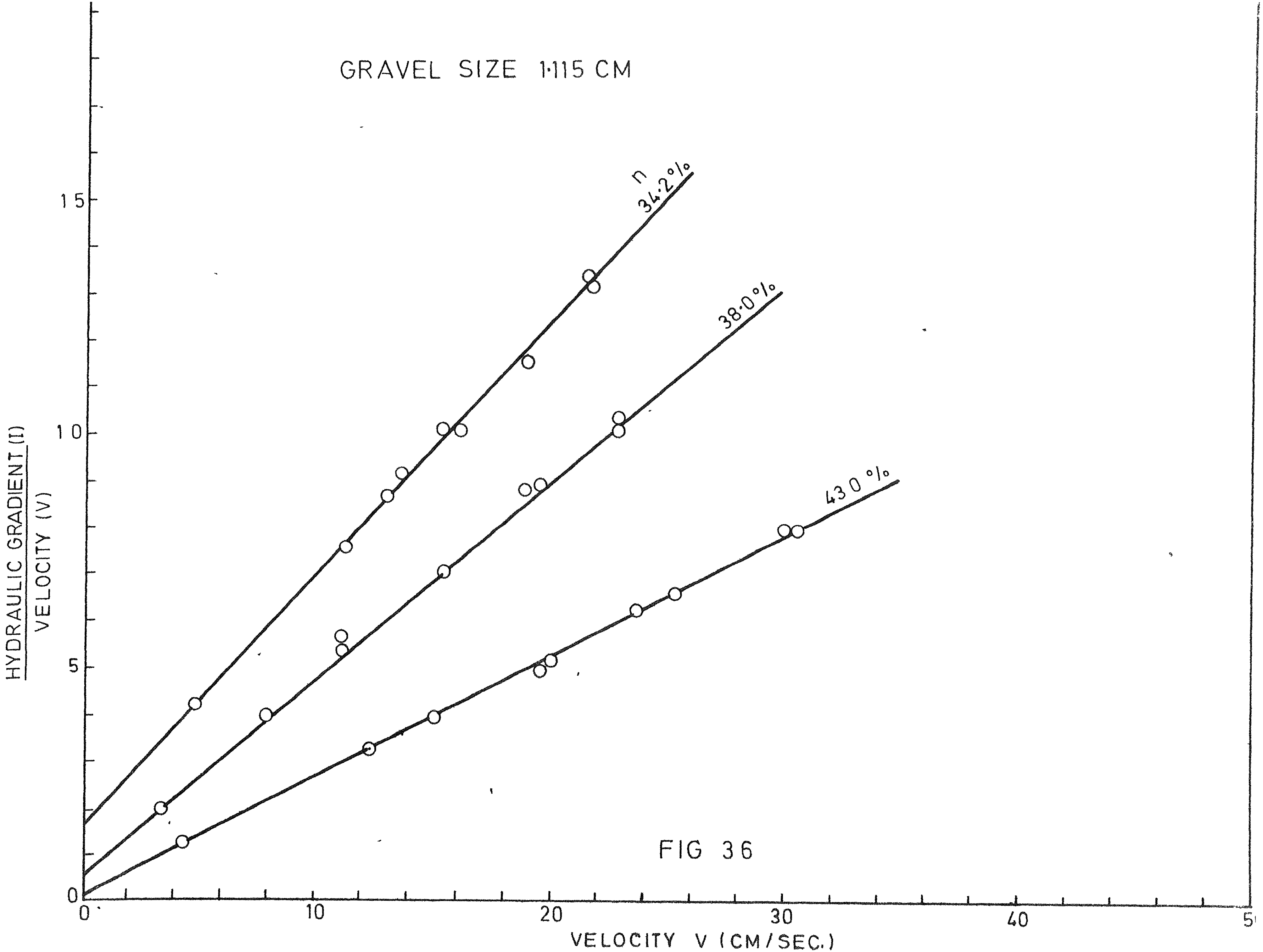

GRAVEL SIZE 0·036 CM
n
36%
40 8 %
43 5 %
HYDRAULIC GRADIENT (I)
VELOCITY (V)
15
10
·5
0
5
10
15
20
VELOCITY V (CM/SEC)

FIG 3·5



GRAVEL SIZE 1·115 CM
n
34·2 %
38·0 %
43 0 %
HYDRAULIC GRADIENT (I)
VELOCITY (V)
0
5
10
15
0
10
20
30
40
50
VELOCITY V (CM/SEC.)

FIG 36

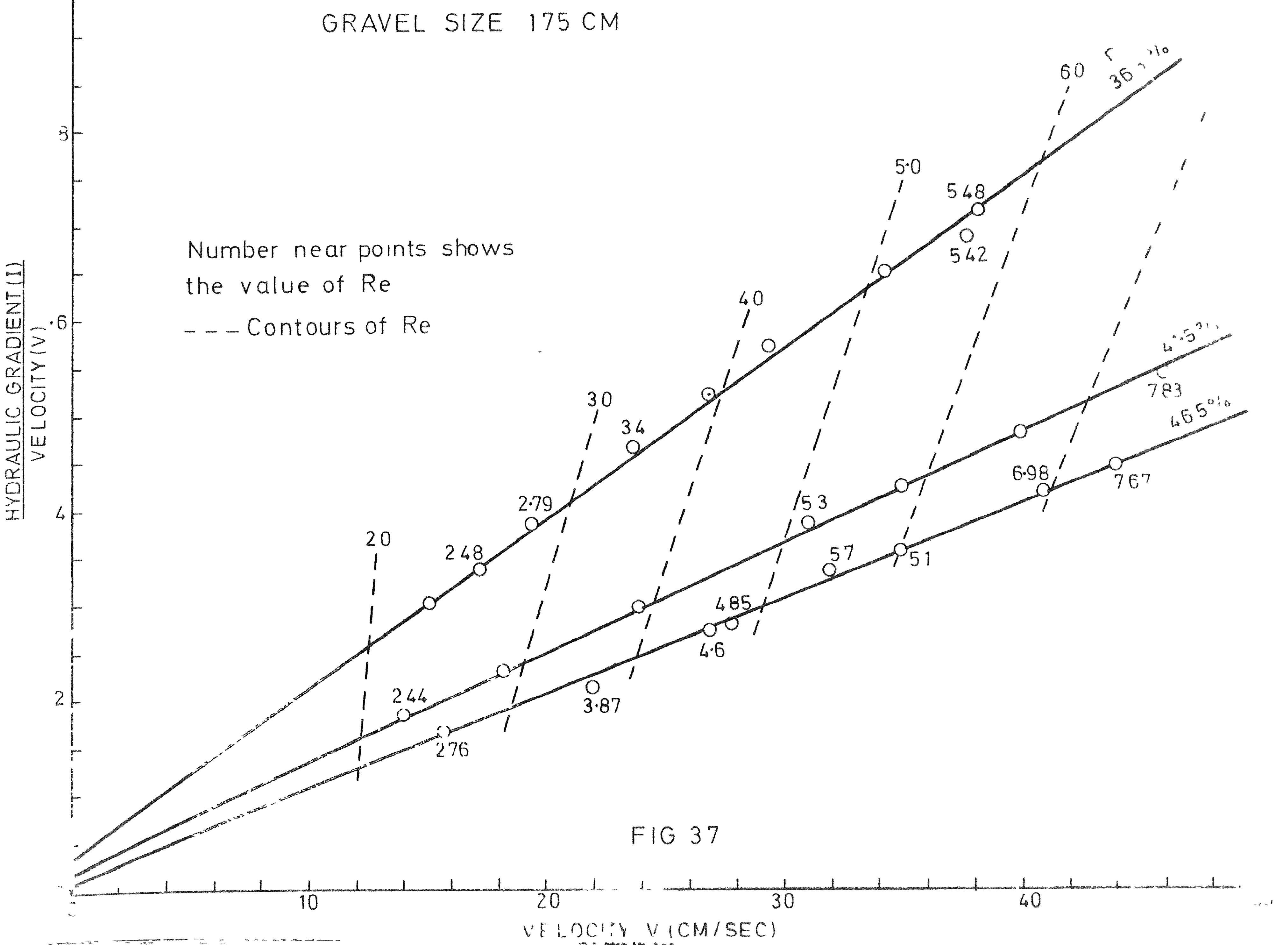
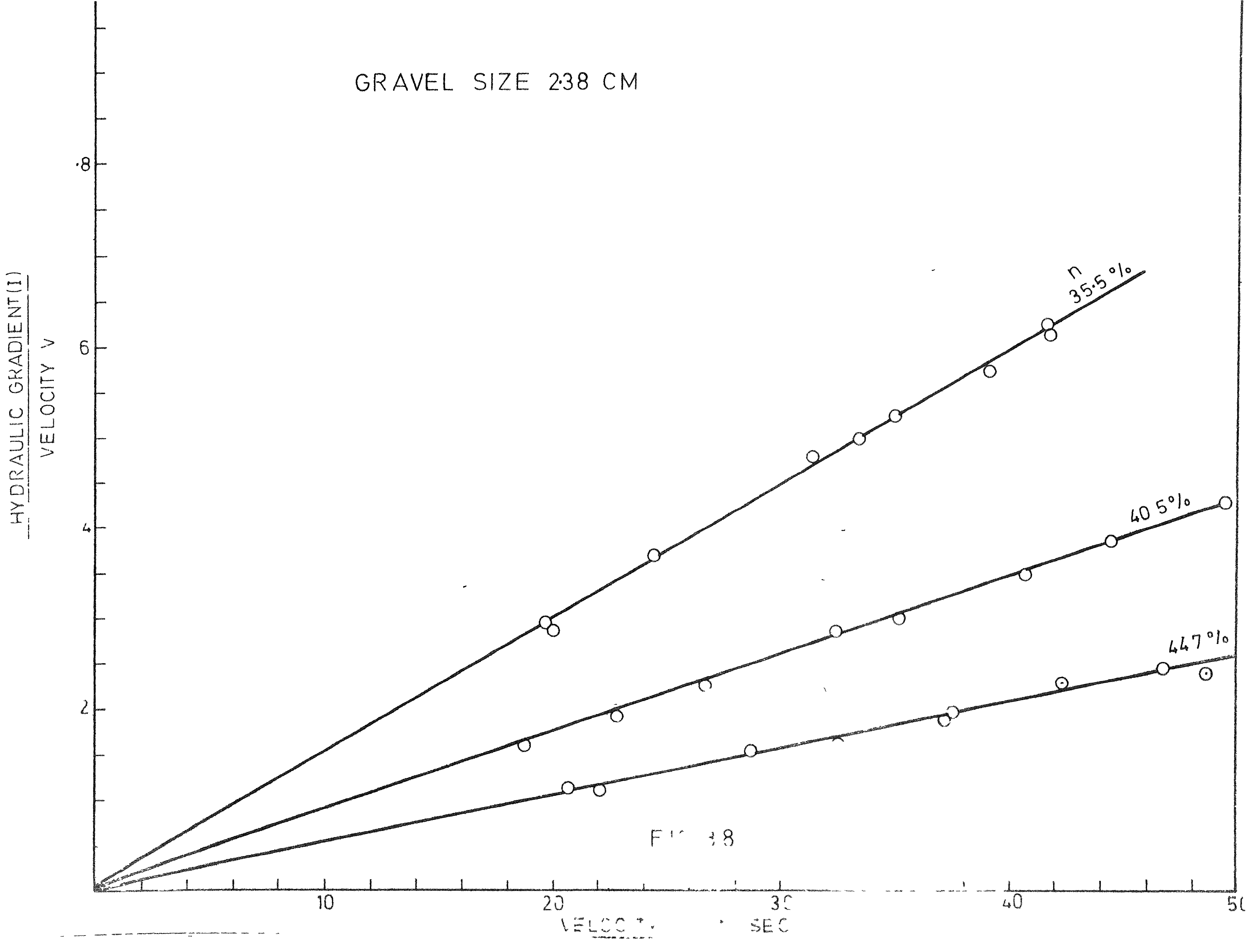

GRAVEL SIZE 175 CM
Number near points shows
the value of Re
- - - Contours of Re
HYDRAULIC GRADIENT (I) / VELOCITY (V)
·6
4
2
VELOCITY V (CM/SEC)
10
20
30
40
20
30
40
5·0
60
2·48
2·79
34
5·48
5·42
2·44
2·76
3·87
4·6
4·85
53
57
51
6·98
7·67
7·83
36
46·5%

FIG 37


GRAVEL SIZE 2·38 CM
HYDRAULIC GRADIENT (I)
VELOCITY V
·8
6
4
2
10
20
40
50
SEC
n
35·5 %
40 5%
447%

GRAVEL SIZE 3·33 CM

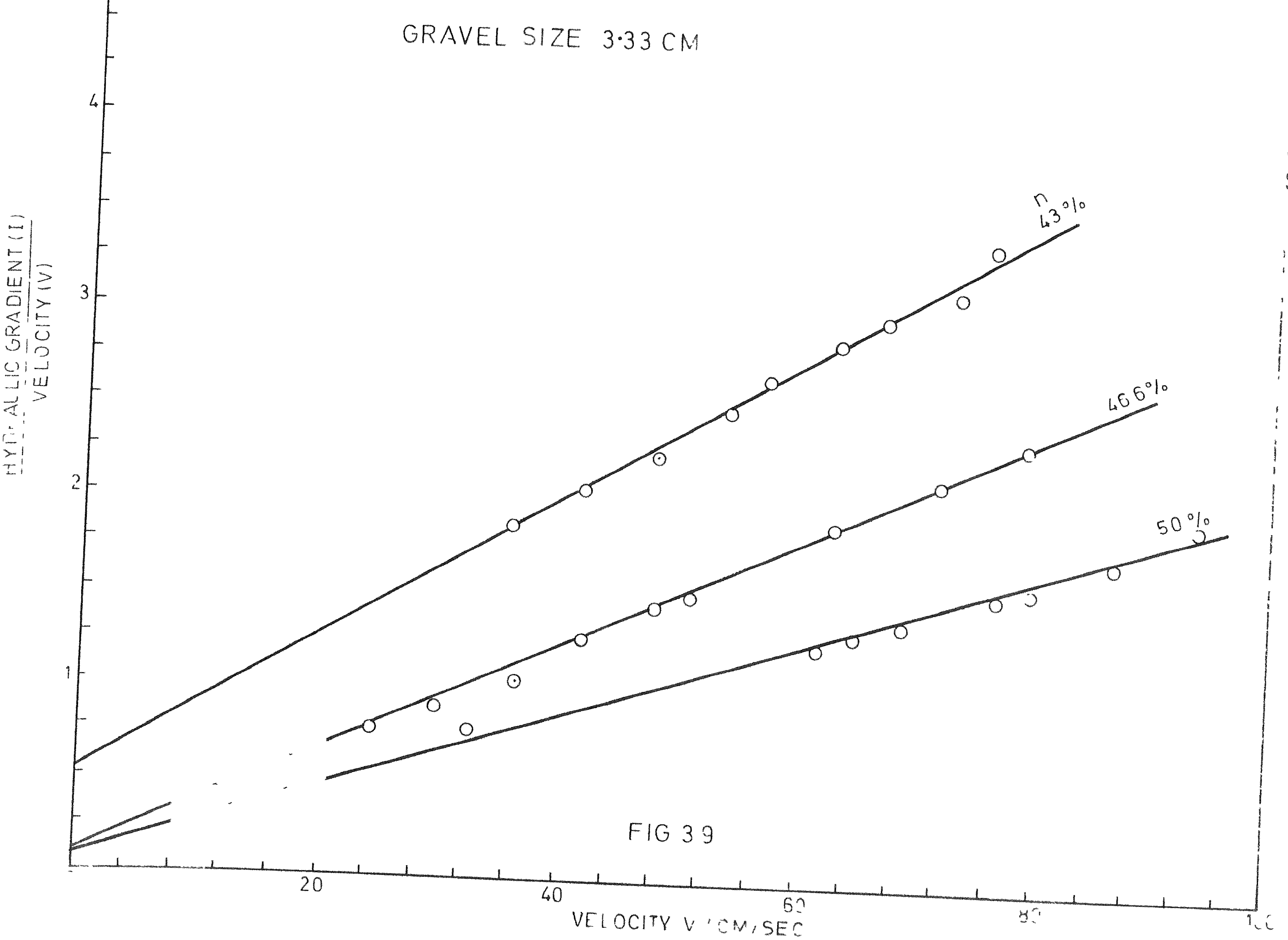
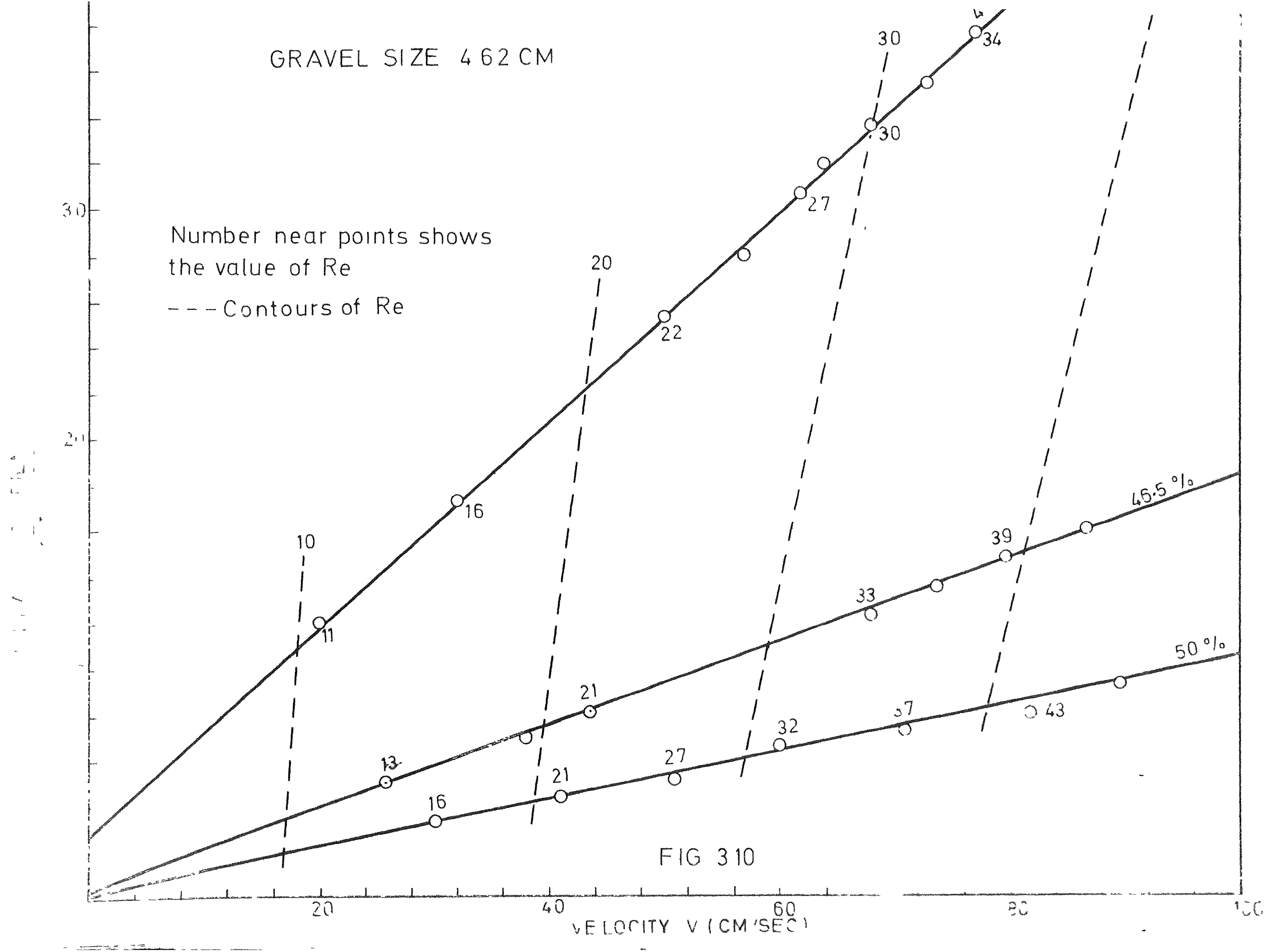
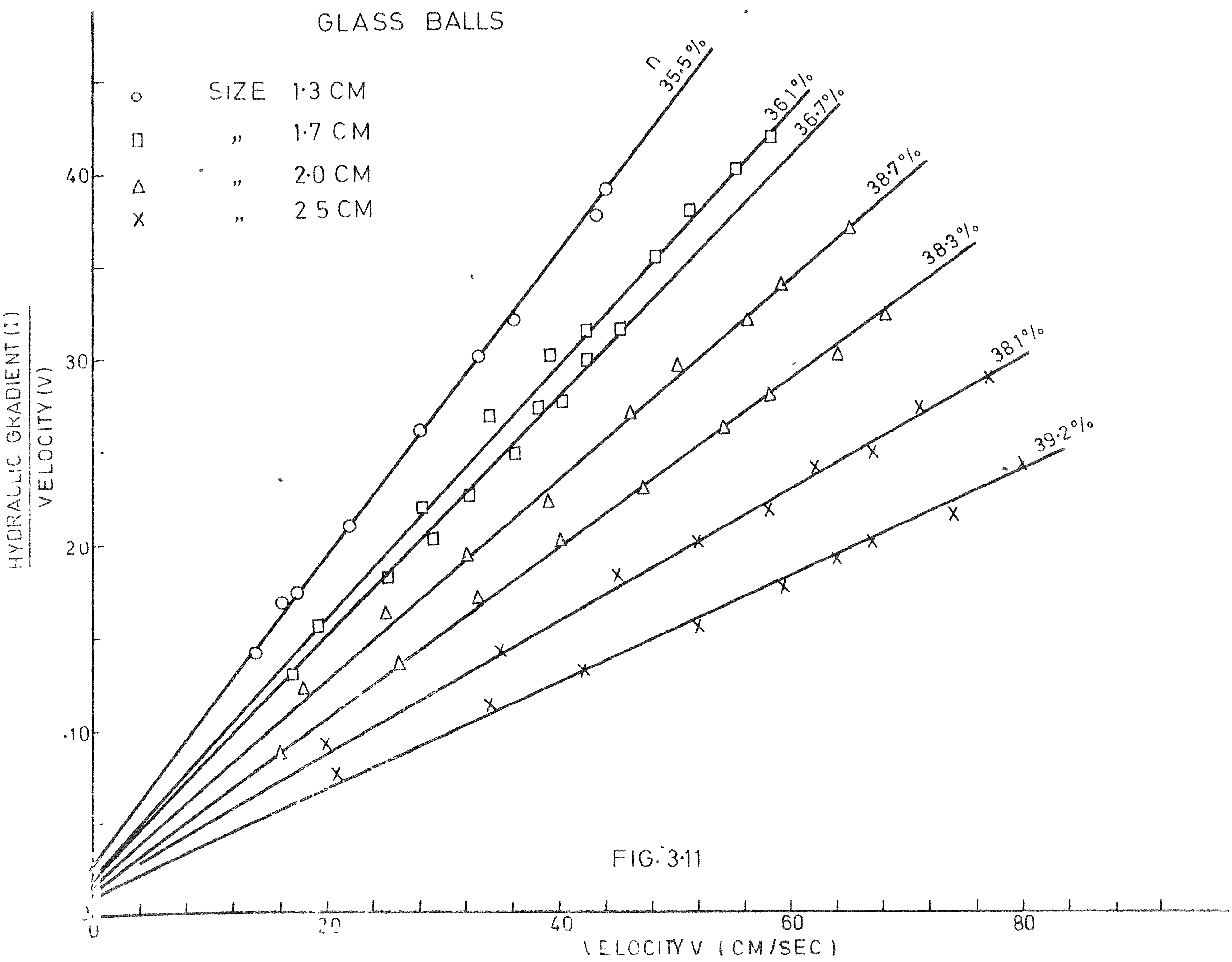


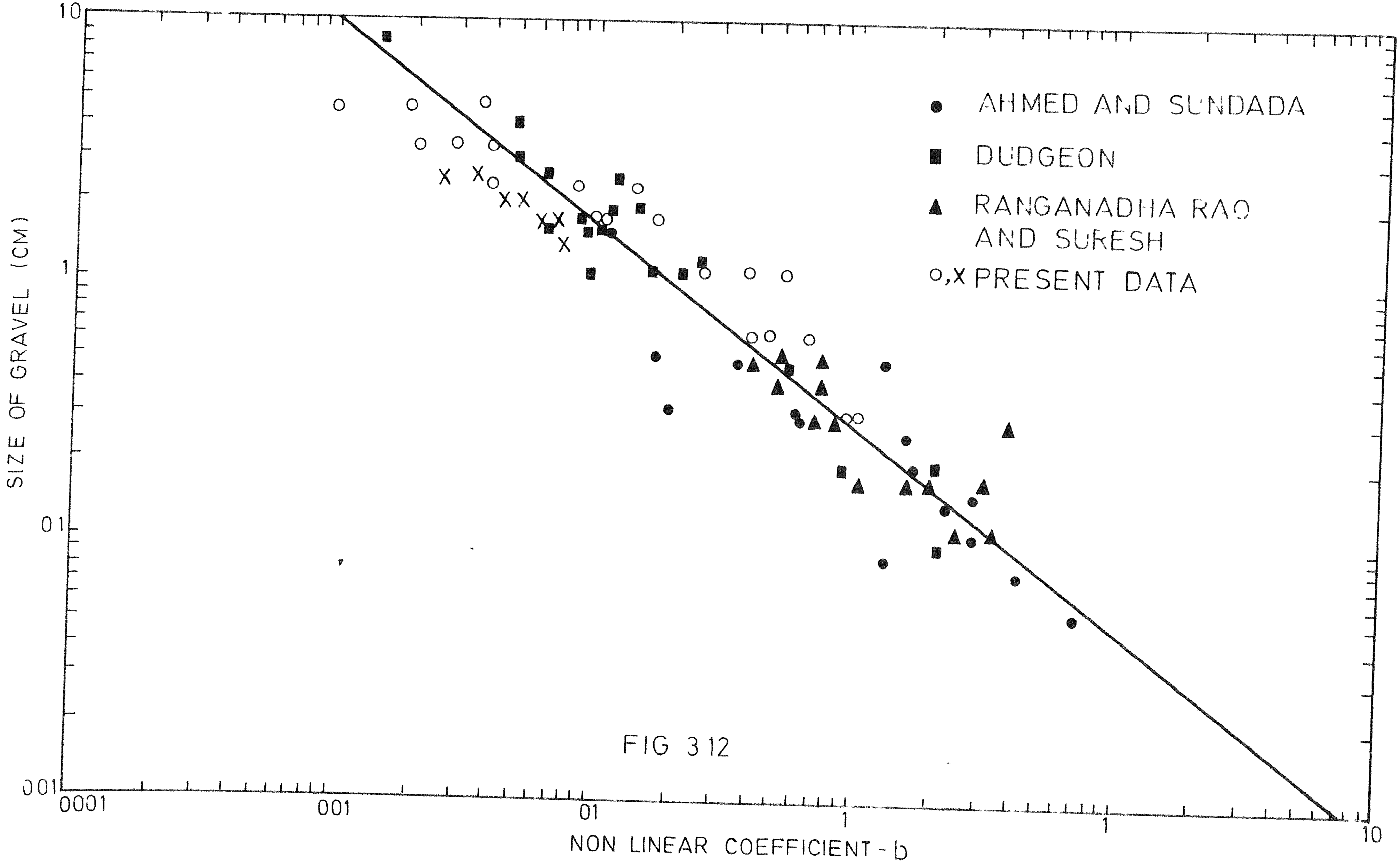
FIG 3 9


GRAVEL SIZE 4.62 CM
Number near points shows the value of Re
- - - Contours of Re
10
20
30
11
16
22
27
30
34
13
21
33
39
46.5 %
16
21
27
32
37
43
50 %
30
20
20
40
60
80
100
VELOCITY V (CM/SEC)
FIG 3.10



GLASS BALLS
SIZE 1·3 CM
" 1·7 CM
" 2·0 CM
" 2 5 CM
n
35,5%
36·1%
36·7%
38·7%
38·3%
38·1%
39·2%
HYDRAULIC GRADIENT (I)
VELOCITY (V)
40
30
20
·10
0
20
40
60
80
VELOCITY V (CM/SEC)

FIG. 3·11



AHMED AND SUNDADA
DUDGEON
RANGANADHA RAO
AND SURESH
o,X PRESENT DATA
10
1
01
001
0001
001
01
1
1
10
SIZE OF GRAVEL (CM)
NON LINEAR COEFFICIENT - b

FIG 3 12

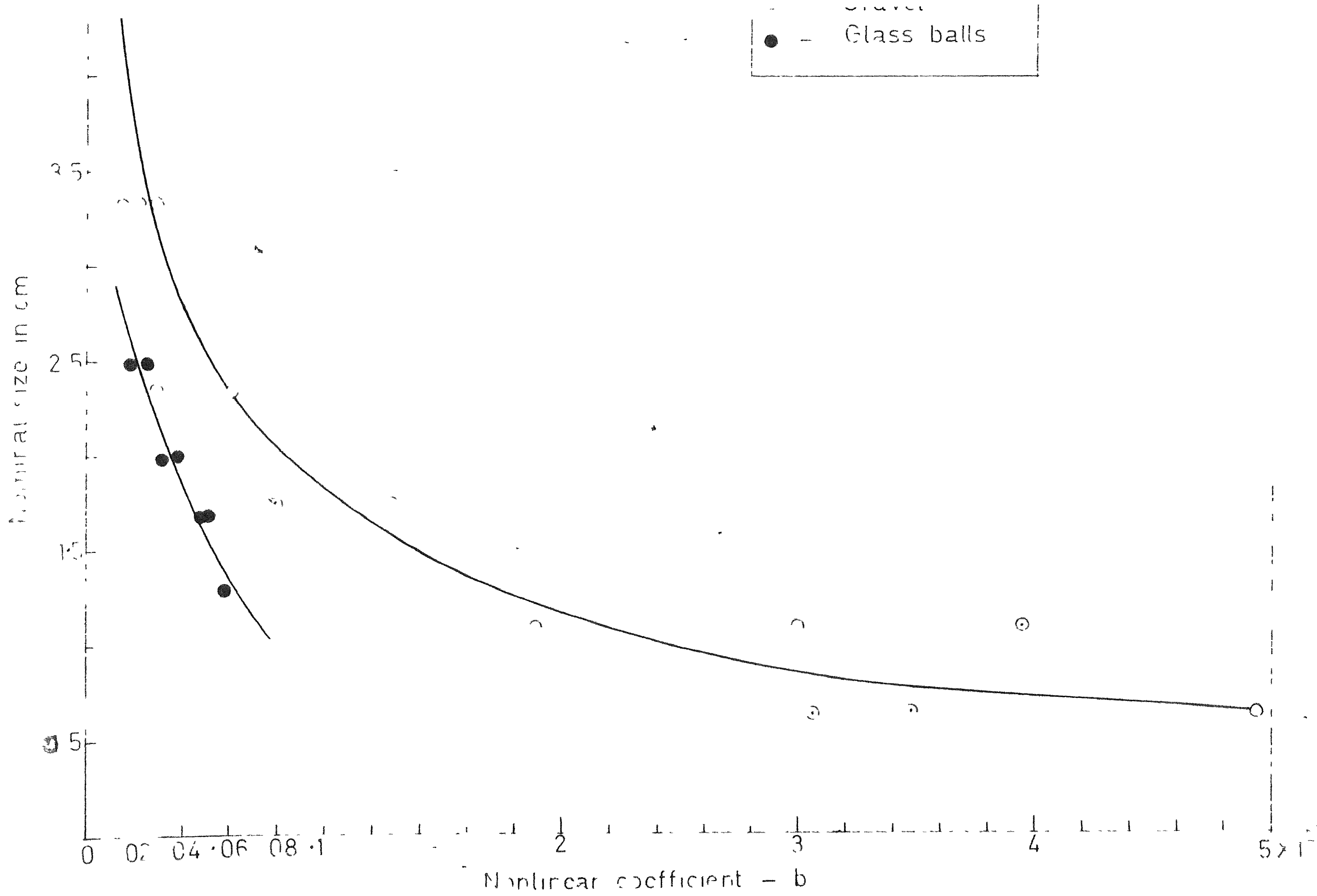


FIG.3.13 VARIATION OF NON-LINEAR COEFFICIENT – b

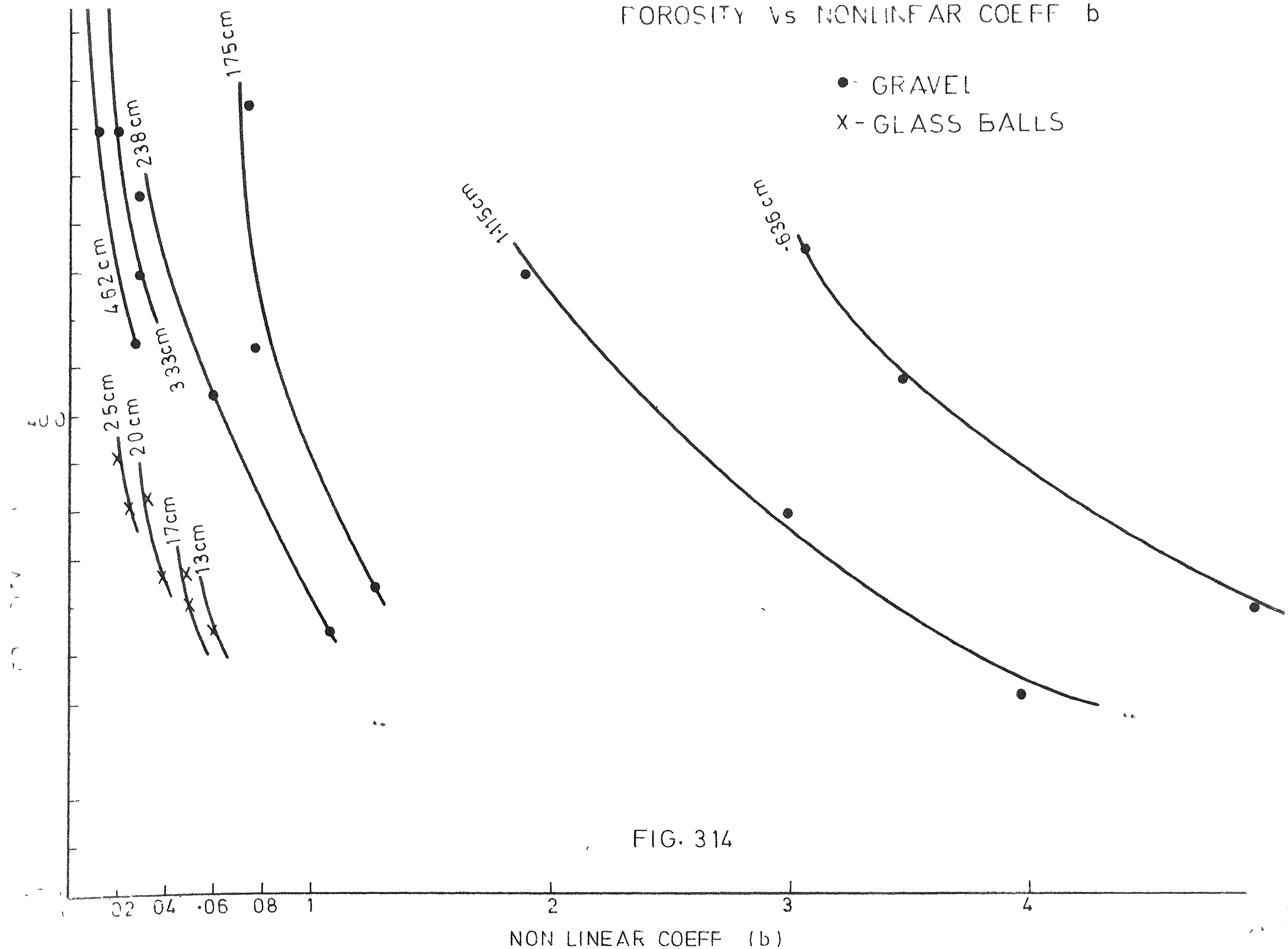
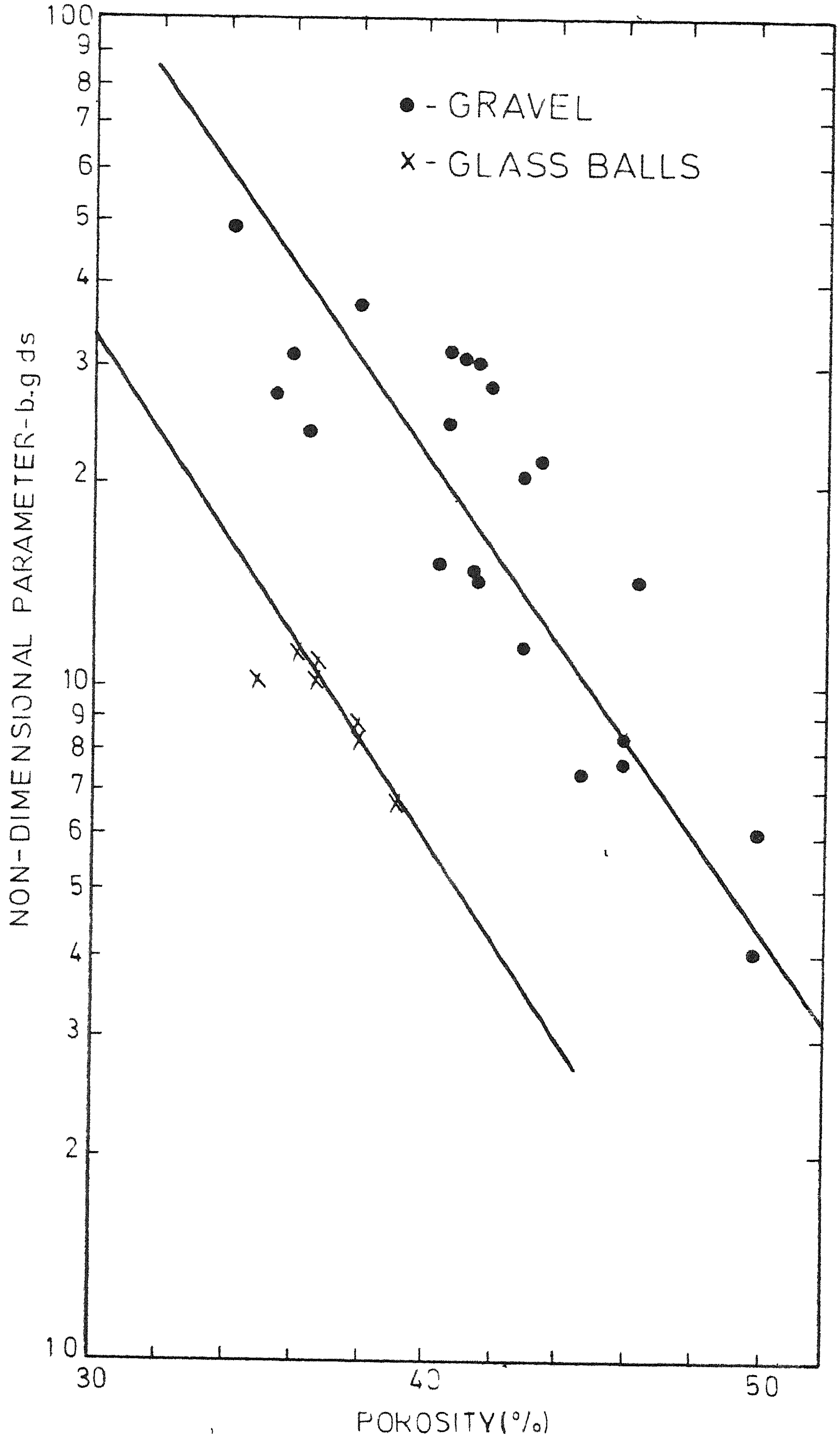

POROSITY Vs NONLINEAR COEFF b
• - GRAVEL
X - GLASS BALLS
175cm
238cm
462cm
333cm
25cm
20cm
17cm
13cm
1·115cm
·636cm
·02
·04
·06
·08
1
2
3
4
NON LINEAR COEFF (b)

FIG. 314



● - GRAVEL
X - GLASS BALLS
NON-DIMENSIONAL PARAMETER-b.g ds
POROSITY(%)
100
9
8
7
6
5
4
3
2
10
1.0
30
40
50

FIG 315

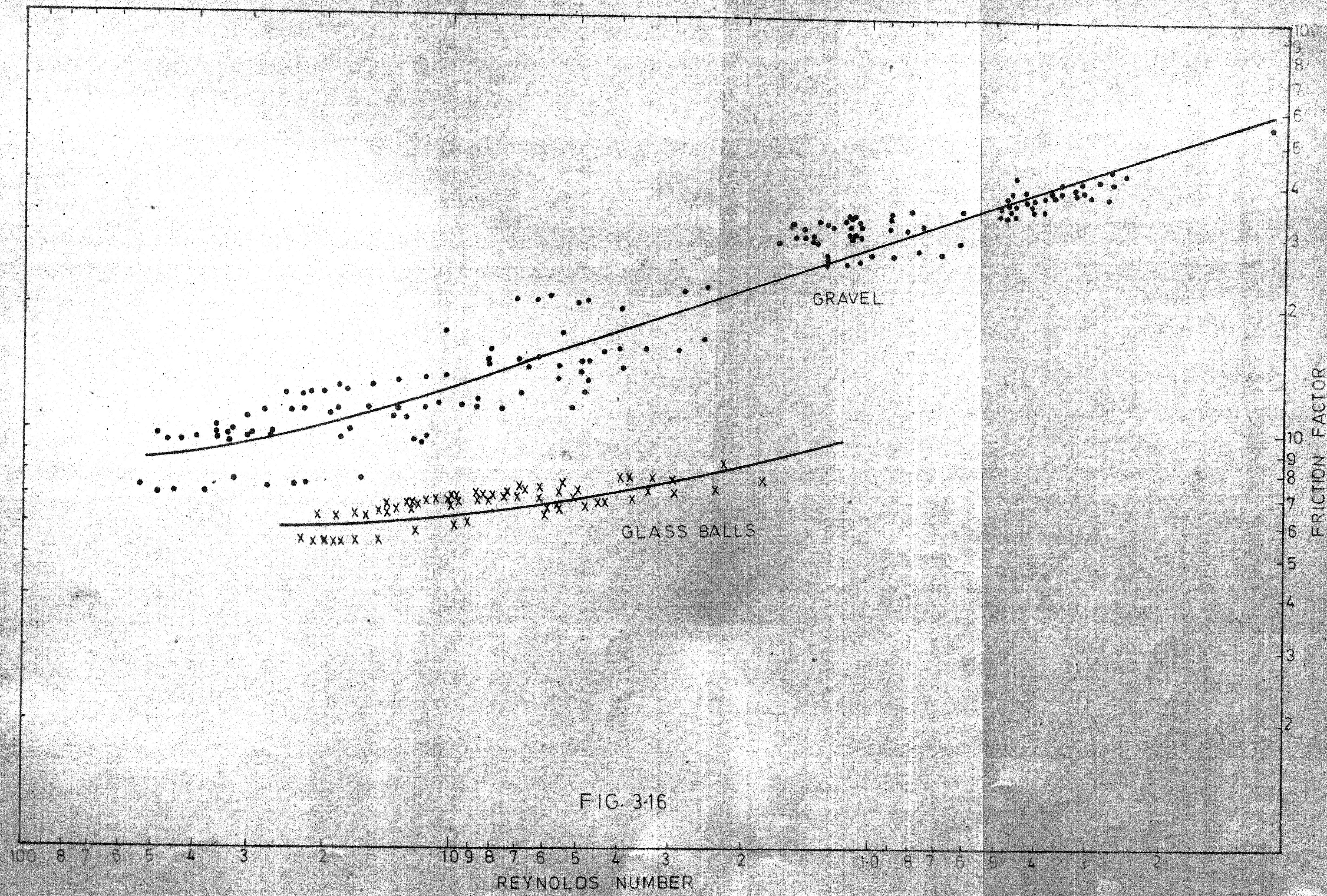

GRAVEL
GLASS BALLS
FRICTION FACTOR
REYNOLDS NUMBER
100 8 7 6 5 4 3 2 10 9 8 7 6 5 4 3 2 1·0 8 7 6 5 4 3 2
100 9 8 7 6 5 4 3 2 10 9 8 7 6 5 4 3 2

FIG. 3·16

CHAPTER 4

DEVELOPMENT OF A CONICAL PERMEAMETER

4.1 GENERAL

In many field situations, flow may be convergent with large variation in hydraulic gradient over a short distance, e.g. flow in to a well, rockfill and earth dams. It has been established that most coarse grained soils exhibit nonlinear flow characteristics. The flow parameter 'K', the coefficient of permeability will not be a constant over the total range of hydraulic gradients. In establishing discharges for such situations, often, permeability is determined using a setup that essentially develops a constant or nearly constant hydraulic gradient.

Therefore, it is felt necessary to develop a new permeameter that can generate variable hydraulic gradients and to test the soils to get some average coefficients from the same. A conical permeameter is constructed. Necessary formula for flow through the same are derived and some preliminary tests were carried out.

4.2 DERIVATION OF THE FORMULAE:

Formulae have been derived for the discharge through the conical permeameter using:

(a) Darcy's law and (b) Forchheimer's equation.

4.2.1 Derivation Using Darcy's Law:-

CASE 1: Constant head case (Fig. 4.1)

r_1 and r_2	are the bottom and the top radii of the conical permeameter.
L	is the height of permeameter
s	is the taper in the sides of the permeameter
h	is the head difference between inlet and exist at any instant.
K	is the coefficient of permeability.
q	is the discharge.

Consider an element dx, at a distance x from bottom, in the permeameter. From Darcy's law

$$q = K\, i_x\, A_x \quad \text{or} \quad i_x = \frac{q}{KA_x} \qquad (4.1)$$

where i_x and A_x are the hydraulic gradient and cross-section area of the permeameter respectively at the strip in consideration

$$A_x = \pi r_x^2 = \pi (r_1 + sx)^2$$

$$i_x = \frac{dh}{dx} = \frac{q}{K\pi(r_1 + s x)^2} \qquad (4.2)$$

or
$$dh = \frac{q}{K\pi} = \frac{dx}{(r_1 + s x)^2} \qquad (4.3)$$

Integrating equation (4.3) and taking limits

$$h = \frac{q L}{K\pi r_1 r_2} \qquad (4.4)$$

or
$$q = \frac{h K \pi r_1 r_2}{L} \qquad (4.5)$$

CASE 2: Variable head case.

If A is the cross-sectional area of the inlet tube, then

$$q = - A \frac{dh}{dt} \qquad (4.6)$$

Equations (4.5) and (4.6)

$$\frac{h r_1 r_2 K \pi}{L} = - A \frac{dh}{dt} \qquad (4.7)$$

Let h change from H_1 to H_2 when time changes from t_1 to t_2.

Separating the variables and integrating equation (4.7)

$$\frac{r_2\, r_1\, K\pi}{L\, A} \int_{t_1}^{t_2} dt = \int_{H_2}^{H_1} \frac{dh}{h} \qquad (4.8)$$

or

$$\frac{r_2\, r_1\, K\pi}{L\, A} (t_2 - t_1) = \ln \left(\frac{H_1}{H_2} \right) \qquad (4.9)$$

Equation (4.9) gives the required relationship. However, if $r = r_1 = r_2$ is substituted in it.

$$\frac{K\, \pi\, r^2}{L\, A} = \ln \left(\frac{H_1}{H_2} \right) \qquad (4.10)$$

which is the formula for cylindrical permeameter.

4.2.2 : Derivation Using Forchheimer's Equation:

CASE 3: Constant Head Case (Fig. 4.1).

Notation remains same. Considering the element dx at distance x from bottom. Applying Forchheimer's equation to this element.

$$i_x = a v_x + b\, v_x^2 \quad (\text{where } v_x = \frac{q}{A_x}) \qquad (4.11)$$

or

$$= a \frac{q}{A_x} + b \frac{q^2}{A_x^2} \qquad (4.12)$$

or

$$\frac{dh}{dx} = \frac{aq}{\pi(r_1 + sx)^2} + \frac{b\,q^2}{\pi^2(r_1 + sx)^4} \qquad (4.13)$$

Separating the variables and taking the limits of equation (4.13).

$$\int_0^h dh = \int_0^L \frac{a\,q\,dx}{\pi(r_1 + sx)^2} + \int_0^L \frac{b\,q^2\,dx}{\pi^2(r_1 + sx)^4} \qquad (4.14)$$

Integrating and simplifying equation (4.14)

$$h = \frac{a\,q\,L}{\pi\,r_1\,r_2} + \frac{b\,q^2 L\,(r_1^2 + r_2^2 + r_1\,r_2)}{3\,\pi^2\,r_1^3\,r_2^3} \qquad (4.15)$$

<u>CASE 4:</u> The Variable Head Case.

Rewriting equation (4.15) as

$$b\,L\,(r_1^2 + r_2^2 + r_1\,r_2)\,q^2 + 3\,a\,L\,\pi\,r_1^2\,r_2^2\,q - 3\pi^2\,r_1^3\,r_2^3\,h = 0 \qquad (4.16)$$

and substituting $P = b\,L\,(r_1^2 + r_2^2 + r_1\,r_2)$

$Q = 3\,q\,L\,\pi\,r_1^2\,r_2^2$

$X = 3\,\pi^2\,r_1^3\,r_2^3$

Equation (4.16) reduces to

$$P\,q^2 + Q\,q - h\,X = 0 \tag{4.17}$$

Roots of equation (4.17) are

$$q = \frac{-Q \pm \sqrt{Q^2 + 4\,P\,h\,X}}{2P} \tag{4.18}$$

Since q can not be negative.

$$q = \frac{-Q + \sqrt{Q^2 + 4P\,X\,h}}{2\,P} \tag{4.19}$$

In variable head case

$$q = -A\,\frac{dh}{dt} \tag{4.20}$$

Equating (4.19) and (4.20)

$$-\frac{Adh}{dt} = \frac{-Q + \sqrt{Q^2 + 4\,P\,X\,h}}{2\,P} \tag{4.21}$$

Let h change from H_1 to H_2 when the change in time is from t_1 to t_2.

Separating the variables and integrating Eq. 4.21

$$\int_{H_2}^{H_1} \frac{A\,dh}{-Q + \sqrt{Q^2 + 4\,P\,X\,h}} = \int_{t_1}^{t_2} \frac{dt}{2P} \tag{4.22}$$

On integrating and simplifying equation (4.22)

$$(A/X)\sqrt{Q^2 + 4\,P\,X\,H_1} - (A/X)\sqrt{Q^2 + 4\,P\,X\,H_2}$$
$$+ (AQ/X)\ \ln\left(\sqrt{Q^2 - 4\,P\,X\,H_1} - Q\right)$$
$$- \frac{AQ}{X}\ \ln\left(\sqrt{Q^2 + 4\,P\,X\,H_2} - Q\right) = t_2 - t_1 \tag{4.23}$$

Equation (4.23) further simplifies to

$$\ln\left[\frac{\sqrt{1 + \dfrac{4\,P\,X\,H_1}{Q^2}} - 1}{\sqrt{1 + \dfrac{4\,P\,X\,H_2}{Q^2}} - 1}\right] = $$
$$= \frac{X\,(t_2 - t_1)}{Q\ A} - \left(\sqrt{1 + \frac{4\,P\,X\,H_1}{Q^2}} - \sqrt{1 + \frac{4\,P\,X\,H_2}{Q^2}}\right) \tag{4.24}$$

The expression $\left(1 + \dfrac{4\,P\,X\,H_1}{Q^2}\right)$ of equation (4.24) on substituting back the values of P, Q and X and simplifying.

$$1 + \frac{4\,P\,X\,H_1}{Q^2} = 1 + B\,\frac{b}{a^2}\,H_1 \tag{4.25}$$

where
$$B = \frac{4}{3L} \left(\frac{r_1}{r_2} + \frac{r_2}{r_1} + 1 \right) \tag{4.26}$$

The equation (4.24) reduces to

$$\ln \left[\frac{\sqrt{1 + \frac{B\,b\,H_1}{a^2}} - 1}{\sqrt{1 + \frac{B\,b\,H_2}{a^2}} - 1} \right]$$

$$= \frac{\pi\, r_1\, r_2\, \Delta t}{L\, A\, a} - \left(\sqrt{1 + \frac{B\,b\,H_1}{a^2}} - \sqrt{1 + \frac{B\,b\,H_2}{a^2}} \right) \tag{4.27}$$

If (b/a^2) is much less than unity, expanding $\left(1 + \frac{B\,b\,H_1}{a^2}\right)^{1/2}$ binomialy and considering the first two terms only

$$\left(1 + \frac{B\,b\,H_1}{a^2} \right)^{1/2} = 1 + \frac{B\,b\,H_1}{2a^2} + \text{Higher power terms} \tag{4.28}$$

Substituting the quantities of (4.28) in equation (4.27) and simplifying.

$$\ln \frac{H_1}{H_2} = \frac{\pi\, r_1\, r_2\, \Delta t}{L\, A\, a} - \frac{B\,b}{2\,a^2} (H_1 - H_2) \tag{4.29}$$

Similar equation can be formed for 2nd and 3rd values of H_2 and H_3, with the condition that Δt remains same.

$$\ln\left(\frac{H_2}{H_3}\right) = \frac{\pi\, r_1\, r_2\, \Delta t}{L\, A\, a} - \frac{B\, b}{2\, a^2}\,(H_2 - H_3) \qquad (4.30)$$

Subtracting (4.30) from (4.29) and simplifying

$$\ln\left(\frac{H_2^2}{H_1\, H_3}\right) = \frac{B\, b}{2\, a^2}\,(H_1 + H_3 - 2H_2) \qquad (4.31)$$

Since there are two unknowns 'a' and 'b', the two equation (4.29) and (4.31) can be used to determine them.

If the value of nonlinear coefficient 'b' is set zero in the equation (4.29), the equation reduces to the equation (4.9) derived from the Darcy's law.

4.3 EXPERIMENTAL WORK:

Coefficient of permeability (K) has been obtained to investigate the effect of variable gradient on it with the help of equation (4.9). To do this three values of 'K' have been obtained, corresponding to three porosities for each type of permeameter and for sand.

The setup is shown in Fig. 4.1. It consist of a permeameter (conical or cylindrical) and a glass jar. For filling the permeameter, required volume and hence the required weight of clean and dry sand was obtained for a particular porosity. This sand was poured-in such that it completely fills the permeameter uniformly. After this cover was bolted assuring no leakage. The inlet tube of the permeameter was then connected to the jar.

Clear water was now poured in to the jar up to the level slightly higher (2-3 cms) than the mark at which the observation was to be recorded. Out let valve was opened and as soon as the water level in the jar crosses the pre-decided mark, the stop watch was started and the heights were recorded after every minute.

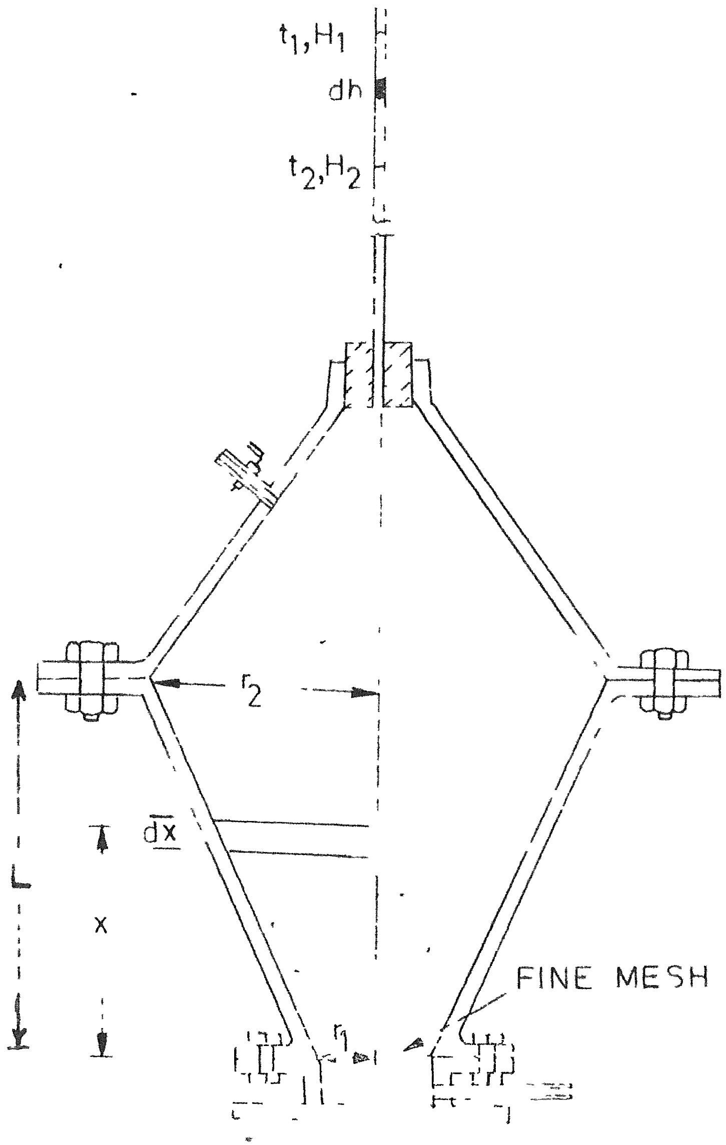
Before taking the observations, the sand was saturated by allowing the water to flow through it for at least 15 minutes. Whole experiment was repeated untill reporducible results were obtained.

4.4 RESULTS

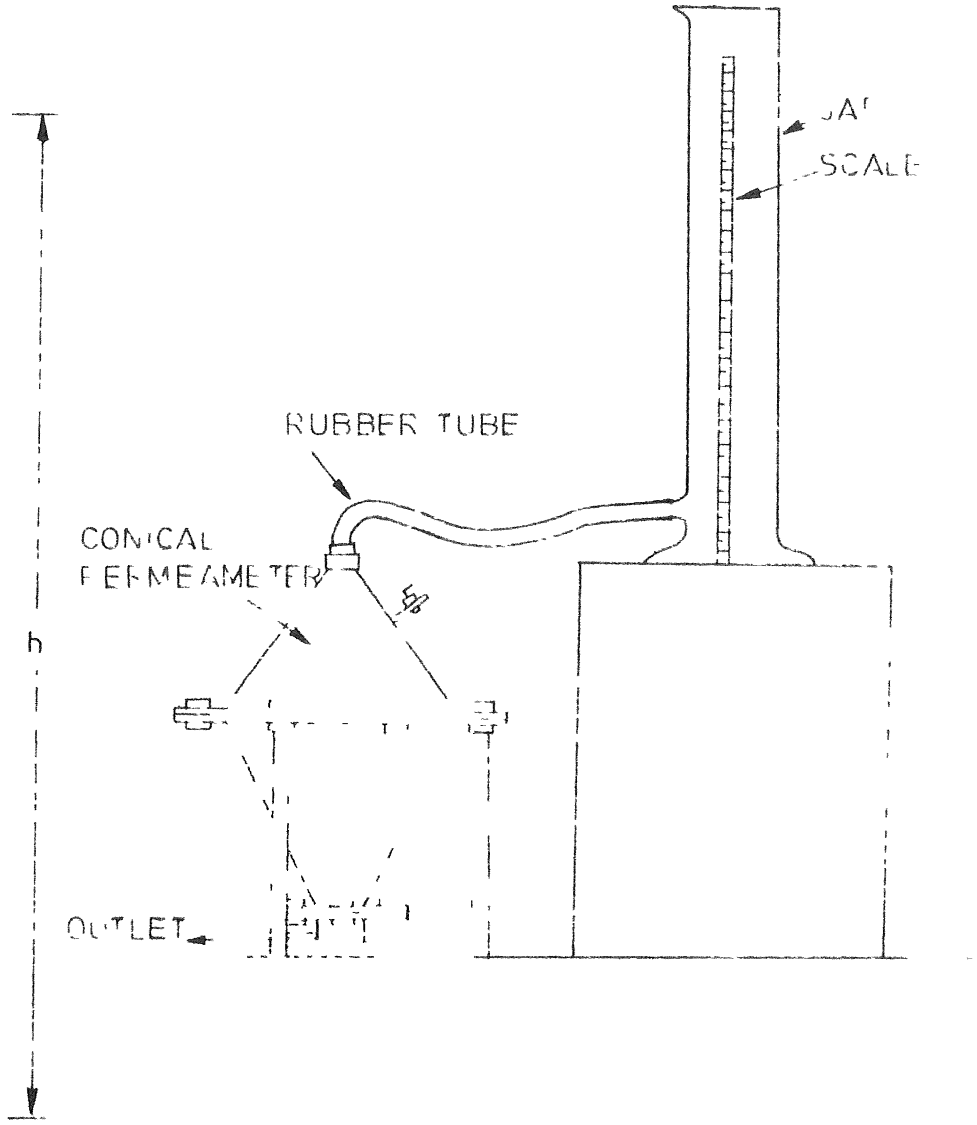
From the recorded observations, the values of 'K' were obtained by using equation (4.9). Thus the values of K, obtained from cylindrical and conical permeameters with the contraction ratio of 1:2 and 1:4, were plotted in

a single plot (Fig. 4.2). A single curve fits all the points, thus establishing that for the range of hydraulic gradients existing in the permeameter, K the coefficient of permeability is nearly independent of the gradient.

Some tests have been conducted and analysis made using equations (4.29) and (4.31). The results are in conclusive and hence are not presented here.

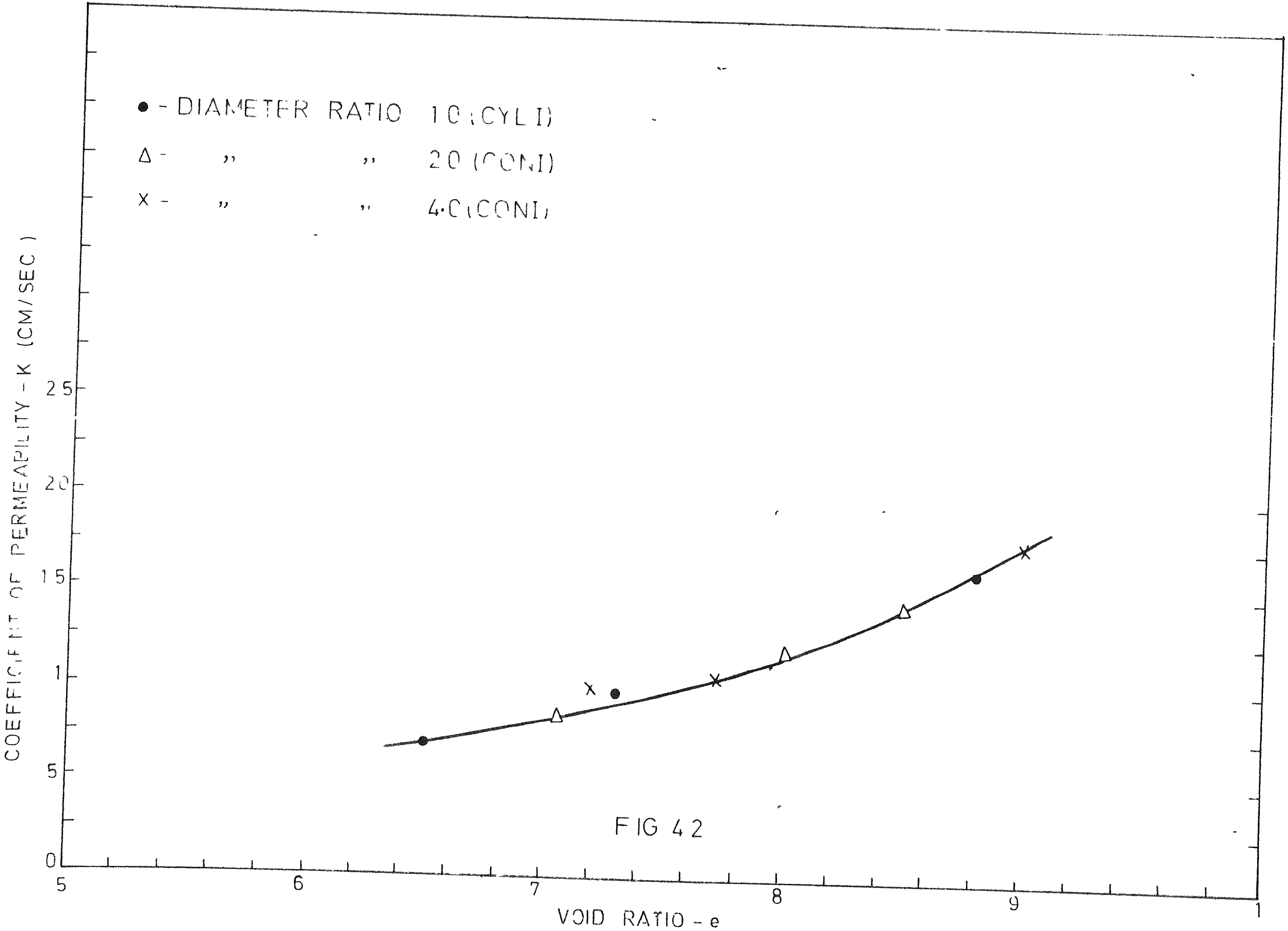


SECTIONAL VIEW



EXPERIMENTAL SET-UP

CONICAL PERMEAMETER



• - DIAMETER RATIO 1·0 (CYL I)
Δ - ,, ,, 2·0 (CON I)
X - ,, ,, 4·0 (CON I)
COEFFICIENT OF PERMEABILITY - K (CM/SEC)
VOID RATIO - e
0
5
1
15
20
25
5
6
7
8
9
1

FIG 4 2

CHAPTER 5

CONCLUDING REMARKS

In this investigation, the flow through soil (porous media) is studied in the 'Steady Inertial Regime' wherein deviations from Darcy's law are observed, but the flow is still laminar. To characterise the velocity hydraulic gradient relation, the Forchheimer equation ($i = av + bv^2$) is chosen in preference to the Missbach equation ($i = cv^m$) as the former is derivable (reducible) from the Navier - Stoke's equations.

The variations in the values of the coefficient 'b' have been studied with size and porosity individually. The effect of surface roughness and shape of the grains could not be seperated and hence are studied in a combined form. It is found that 'b' decreases with increasing size as well as porosity and the variations between them have been shown in Fig. 3.12 and 3.14. These graphs also depict the effect of surface roughness and shape of gravel. A relation-ship has been developed between coefficient 'b' and the size (d) of gravel, which follows:

$$b = \frac{0.0165}{d^{1.41}} \qquad (5.1)$$

From the above equation an approximate estimate of the 'b' value may be obtained for a given size. Another relationship has been proposed for calculating a fairly accurate value of the coefficient 'b' in which the effect of size (ds), porosity (n), shape and surface roughness (R') of the gravel have been combined together. The relationship is as follows:-

$$b = \frac{1}{(ds\ n)\ R'} \qquad (5.2)$$

In the above equation parameter R' is 1.0 for angular and rough surfaces and is 1.25 for spherical and smooth surfaces. A new nondimensional parameter (h.g.ds) has been obtained and its variation with porosity has been shown in Fig. 3.15. This graph also shows the effect of shape and roughness of the gravel and may be used to ascertain the coefficient 'b'. Eventhough a relation appears to exist between the non-dimensional parameter (b.g.ds) with porosity (n), the points obtained are few to arrive at the relationship. A large number of tests need to be carried out further.

A curve (Fig. 3.16) between friction factor and Reynold's number has been drawn in which points are found to be scattered. The reason for such scattering of points is the range of porosity in which each size has

been tested. This is also obvious from the fact that for different materials at a particular Reynold's number (Re), friction factor (F) can not be the same since it is a function of particle size, shape, surface roughness and porosity.

In any field situation, the hydraulic gradient is not constant but varies in space the coarse grained soils (like sands, gravels etc.) exhibit nonlinear relation between hydraulic gradient and velocity. So it was felt necessary to develop a permeameter which can generate a range of hydraulic gradients and get an average coefficient of permeability for the range generated conical permeameter is one such which satisfies the above requirement. Expressions have been derived for head loss and discharge through the conical permeameter for either linear (Darcy's equation) or non linear (Forchheimer's equation) flow conditions. Some typical results for K are obtained for fine sand which compare well with those obtained using a cylindrical permeameter.

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